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THE UNIVERSITY OF ALBERTA

INVESTIGATION OF FOUNDATION DEFORMATION USING IN SITU PRESSURE PROBE

by

©

NORMAN ALEXANDER MORRISON

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH

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THE UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES AND RESEARCH

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research, for acceptance, a thesis entitled INVESTIGATION OF FOUNDATION DEFORMATION USING IN SITU PRESSURE PROBE submitted by Norman Alexander Morrison in partial fulfilment of the requirements for the degree of Master of Science.

ABSTRACT

In the case of overconsolidated soils and soft rocks, with water content at or below saturation level, the undrained component of foundation deformation is a substantial percentage of the total deformation. Som (1968), upon studying the behavior of structures founded on the overconsolidated London clay, observed that the undrained settlement constituted on average 57.5% of the 50 year settlement. DeJong (1971) observed similar foundation deformation behavior both with respect to settlement and heave response. DeJong's research involved the partially saturated and saturated overconsolidated soils and soft rock that make up the foundation material of central Edmonton.

The theory of elasticity can be used to estimate the undrained component of settlement or heave if representative deformation moduli can be obtained. The deformation modulus is recognized as being very sensitive to sample disturbance effects, with modulus values obtained in the laboratory usually being much lower than would be supported by field observations. For example DeJong (1971) concludes from his research that "the deformation moduli obtained from conventional laboratory tests proved to be unreliable when compared to the field moduli," and "the ratios of field to laboratory moduli are large indicating that the use of laboratory moduli will lead to gross over-estimates of settlement". Therefore in order to make accurate predictions of the undrained deformation of foundations, a reliable method of obtaining representative deformation moduli is necessary.

Based upon this philosophy in situ pressuremeter tests were performed to determine the deformation moduli of the overconsolidated foundation strata underlying the CN and AGT Towers in Edmonton. A depth

of 170 feet and 190 feet were investigated at the CN and AGT Tower sites respectively.

DeJong (1971) had obtained very good load-settlement data for the CN Tower and excavation-heave data from the AGT Tower excavation. The availability of this data along with the high percentage of undrained or immediate deformations experienced at both these locations, provided an excellent opportunity to determine the acceptability of the pressuremeter as a device for obtaining the in situ deformation moduli of overconsolidated soils and soft rock.

The values of deformation moduli obtained from the in situ pressuremeter tests were used in two and three-dimensional finite element analyses. The results obtained for the settlement of the CN Tower and the base heave of the AGT Tower excavation, agreed remarkably well with actual field observations.

The adequacy of the analytical technique assisted in demonstrating the accuracy with which the in situ pressuremeter test can determine the actual deformation modulus of overconsolidated soil and bedrock.

An account of the present inadequacies of the pressuremeter probe and the economics involved in this type of foundation investigation are also discussed.

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CHAPTER I

INTRODUCTION

1.1 Theory

Deformations in soil and rock as well as in other media, arise from strains produced by a change in normal and shear stresses. A change in the state of stress of a soil or rock will result in total deformations that consist of immediate and time-dependent components.

The immediate deformation of soil or rock upon change in applied total stress is known also as the undrained component of deformation. In the case of saturated ground this refers to shear deformation under no volume change. For partially saturated ground, immediate deformation refers to shear deformation under undrained conditions but also includes volume change deformation insofar as there is an immediate normal effective stress response giving rise to volume deformation.

Deformations arising from volume change produced upon dissipation of excess pore pressure are known as primary consolidation deformations and are time-dependent. The excess pore pressure generated is a function of the change in normal stress and the change in shear stress (Skempton, 1954). The extent to which a change in normal stress contributes to the generation of excess pore pressure is a function of the degree of saturation of the soil or rock mass. A change in shear stress affects the pore pressure insofar as a change in volume of the soil mass may accompany the change in shear stress. Volume change of the soil mass due to shear stressing is a function of the type of soil and its stress history.

A further time-dependent deformation occurs under essentially constant effective stress and is referred to as secondary consolidation deformation.

The portion that each of these deformation components contributes to the total deformation is primarily a function of the type of soil foundation and its stress history. It has been observed by Som (1968) and DeJong (1971) that structures founded on overconsolidated soil foundations generally have lower total settlements than those located on normally consolidated soil. This they attribute to the decreased compressibility of the overconsolidated soil arising as a result of pre-stressing. Som (1968) and DeJong (1971) further observed that the immediate component of settlement is much greater for overconsolidated soils than for normally consolidated soils, usually in the order of 60% of the 50 year settlement, as opposed to about 16% in the case of normally consolidated soils.

The explanation offered by Som (1968) for experiencing much higher immediate deformations in overconsolidated soils than normally consolidated soils under identical conditions of loading is that the pore pressure parameter A (Skempton, 1954) is much lower for overconsolidated soils than for normally consolidated soils (Bishop and Henkel, 1962). Therefore the excess pore pressure generated in the overconsolidated soil foundation is considerably less (Skempton and Bjerrum, 1957) and consequently the immediate component of deformation is increased and the consolidation component is decreased.

1.2 Scope of the Thesis

The foundation materials in the Edmonton area are highly overconsolidated and therefore according to Som (1968) and DeJong (1971) the total deformations of structures founded on these soils will be small and consist largely of the undrained deformation component. Therefore it is

of great importance to be able to predict the undrained settlement or heave in this case.

The theory of elasticity can be used to predict immediate or undrained deformations of soils subjected to a change in the state of stress. However it is necessary to obtain the undrained modulus of deformation of the soil in order to carry out any calculations based on the theory of elasticity. Unfortunately it is a well established fact that the deformation modulus of a soil is very sensitive to soil disturbance and attempts to measure it in the laboratory are usually unsuccessful.

This thesis presents an attempt to measure the modulus of deformation of the overconsolidated soils at two locations in downtown Edmonton (CN and AGT Towers).

The deformation modulus was measured using the in situ pressuremeter probe and Pitcher samples were taken from adjacent boreholes for comparative laboratory testing.

Load-settlement data of the CN Tower and excavation-heave data from the AGT excavation was provided by DeJong (1971).

The geology at both tower locations consisted of distinct relatively flat lying strata. The heterogeneous nature of this foundation necessitated the use of finite element deformation analysis for the prediction of the undrained component of deformation.

CHAPTER II

ELASTIC DEFORMATION OF FOUNDATIONS

2.1 Introduction

Problems encountered in geotechnical engineering are primarily concerned with settlement and stability. Settlement of structures founded on soils is comprised of three physically different soil mass responses induced by a change in total stress. These three components of settlement are initial or undrained reaction, primary consolidation, and secondary consolidation. For the case of saturated soils, initial deformation refers to the response of the soil to change in total stress under undrained conditions and the consolidation components refer to deformation as the result of drainage. Soil permeability and rate of loading chiefly determine the degree of overlap between settlement phases, with the relative magnitudes being a function of soil type, depth of compressible layer, and degree of overconsolidation (Hanna, 1953; DeJong, 1971).

The undrained deformation component is of central concern to this thesis insofar as it is the major contributor to total deformations in overconsolidated soil foundations (Som, 1968; DeJong, 1971).

Particular attention is given to the pressuremeter as a device for measuring the deformation modulus and undrained strength.

2.2 Elastic Analysis Techniques

Stress-displacement analyses have been the object of much research in the past, but not until the widespread availability of computers, have

[illegible]

the methods gained much versatility.

Boussinesq, in the 19th century, developed a closed form solution for deformation of a point load on the surface of a homogeneous, isotropic, linearly elastic half-space. Steinbrenner, Newmark, Terzaghi and others developed various integration procedures, based on Boussinesq, for the solution of more practical boundary condition problems (Terzaghi, 1943). The basic assumptions underlying these solutions, made it necessary to idealize soil and loading conditions often to the extent that the answers were unacceptable in practice.

To further the application of these elastic solutions, for irregular loading conditions, subsequent research moved toward developing influence values which were incorporated into the elastic equations.

Also necessary for the application of the elastic solution is a representative deformation modulus, the measurement of which remains a problem to this day.

Lambe (1964) combined elastic and consolidation deformations with his effective stress-path method and in one analysis, provided one of the first pragmatic treatments of stress-deformation behavior. Pleasing in principle, inherent is the notion that "strain, porewater pressure, and strength of a soil element depend on stress path" (Lambe, 1967). One problem with this method is its tendency to become indeterminate as the loading and soil conditions gain in complexity (Lee, 1968). A circular state arises where, in order to define the stress path of a particular soil element, prior knowledge of the soil behavior is necessary. Observations by Ladd (1964), Lo et al. (1971) and others indicate soil sample disturbance to be the greatest single factor working against this

and other analytical techniques depending upon laboratory moduli.

The finite element technique is the latest numerical procedure that can be used for stress analysis and was the one employed in this thesis. Zienkiewicz (1971) summarily defines the finite element method as "essentially a process through which a continuum with infinite degrees of freedom can be approximated to by an assemblage of subregions (or elements) each with a specified but now finite number of unknowns".

The technique requires a computer for its mathematical operations, but has the versatility of dealing with nonlinearity, anisotropy and non-homogeniety, under any loading and displacement boundary condition. However defining the stress-strain properties that go into the analysis still remains a problem (D'Appolonia et al., 1971).

2.3 Measurement of Stress-Strain Relationships in Soils

(a) Introduction

Although the analytical techniques have reached a high degree of sophistication, the accurate measurement of the in situ stress-strain characteristics of soils has not developed comparably.

The particulate nature of the solid component of soil makes the stress-strain behavior of this material exceedingly complex. Even the concept of stress as it applies to soils is not straight forward in that the interparticle forces on any given plane are not continuous and therefore a macroscopic definition of stress is required. Similarly strain is usually treated at the macroscopic level since the sliding, rolling and distortion that takes place at the microscopic level is complicated and difficult to account for in a stress-strain analysis. The presence of pore fluid in most soils, further complicates stress-strain behavior by

introducing the dimension of time into the state of effective stress and strain.

Despite the peculiar nature of soil, the theory of elasticity can be used with success for the prediction of undrained deformations in soils. In adopting this theory one assumes that stress is a function of strain. Most of the useful solutions from the theory of elasticity also assume that the soil is homogeneous and isotropic.

Although in applying the theory of elasticity to load-deformation problems in soils it is necessary to idealize the soil structure, the biggest difficulty has been the determination of a representative modulus of deformation of the soil. The accurate determination of the modulus of deformation (E) was the prime objective of the foundation investigations discussed in this thesis and has also been the object of much concern in general (Ladd, 1964; Lo et al., 1971; and Milovic, 1971).

(b) Tests for Undrained Deformation Modulus

Deformation moduli are determined from static or dynamic loading tests, in situ or on laboratory samples depending upon the nature of the field loading condition and the economics of the project.

Static tests on laboratory samples usually take the form of unconfined compression and undrained triaxial testing in which the modulus is measured directly, (Ladd and Lambe, 1963; Ladd, 1964; and Lo et al., 1971) or indirectly by measuring strength (Skempton and Henkel, 1957).

Laboratory values tend to grossly underestimate E and consequently greatly overestimate settlement. Modulus values are recognized as being dependent upon many factors of varying importance, with parametric studies being performed to identify their relative influence.

Ladd (1964), on moduli from UU and CIU tests on clay, found the level of applied shear stress, degree of overconsolidation, rate of shear, sample disturbance and confining pressure to be strong influencing factors. His CIU tests at in situ effective stress, gave higher moduli than the UU tests due to the removal of some sampling disturbance effects by consolidation.

Lo et al, (1971) reports no appreciable effect of strain rate and sample size on UU moduli of stiff fissured clay but names sample disturbance as the greatest contributor to low laboratory modulus values (also observed by Crawford, 1963).

Bjerrum and Lo, (1963) and Ladd, (1964) found a direct variation in modulus with time allowed for secondary consolidation prior to undrained shear. It is suggested that at least one log cycle of secondary consolidation be permitted to "re-age" the soil.

Some examples of laboratory and field moduli discrepancies are listed below in Table 2.1. In comparing the respective values one should keep in mind the amount of interpretation required in arriving at a field modulus, in particular, the accuracy with which the stress distributions, loads, and settlements are known.

TABLE 2.1

TYPICAL FIELD/LABORATORY MODULUS RATIOS

$\frac{E \text{ (field)}}{E \text{ (lab)}}$	Lab Test	Project	Report
4 to 5	unconfined compression	Buildings (Boston Area)	Liepens (1957)
4 to 5	unconfined compression	General Discussion	Bjerrum (1958)
3 to 15	undrained triaxial	Excavation Heave	Serota & Jennings (1959)
1*	unconfined compression	Excavation Heave - Ottawa Sewage Treatment Plant	Bozozuk (1963)
4 to 13	unconfined compression	Settlement - Toronto Mt. Sinai Hospital	Crawford & Burns (1963)

*from best tests run on block samples

Lo et al., (1971) report block sample modulus values to be from 4 to 7 times greater than Shelby tube sample moduli.

An attempt to minimize the effects of sample disturbance is outlined by Ladd (1971). Known as SHANSEP (Stress history and normalized soil engineering properties), the main difference from standard procedures is a reconsolidation of the sample to an effective stress greater than in situ; the notion being that disturbed samples if properly reconsolidated in the laboratory, can achieve "soil structure" and normalized properties that are very similar to those of the in situ soil.

This testing method gives better modulus values but the necessity of high pressure triaxial equipment and the uncertainty of the "proper" overconsolidation ratio to apply detracts from the procedure.

Laboratory and field dynamic testing for modulus has its principal use in determining a response to cyclic and impact loading, and is not of concern in this thesis. Dynamic E values have often been used as a guide to rock quality in static loading projects (Dixon, 1968). The concern of this thesis is the measurement of the static modulus.

In situ static modulus tests common in practice are the plate bearing and borehole pressure devices. The moduli obtained in situ are generally much greater than the laboratory moduli determined from borehole samples (Higgins, 1969; and Lo et al., 1971), and often are in good agreement with moduli determined from observations of foundation performance.

Plate bearing tests provide acceptable moduli by testing relatively undisturbed soil, but the physical nature of the test limits its versatility. A single surface test is generally inadequate if the soil foundation is deep, compressible, non-homogeneous, and where the loading is of considerable areal extent. Plate bearing tests have been performed down a borehole with success (Burland and Lord, 1969) and at a reasonable cost (Lake and Simons, 1969) and so if necessary a profile of modulus with depth can be obtained with plate bearing equipment.

The Menard type pressuremeter is (to the author's knowledge) the only borehole device commonly used at present in soil to measure its deformation properties. According to Menard (1965), the first pressuremeter was conceived by Koegler in Germany at an earlier time but he was unable to devise a workable mechanical system.

Menard in 1957 re-invented the pressuremeter device which he used in graduate studies at the University of Illinois. It has seen extensive use in Europe in the past 15 years but only recently has it come into general use in North America (Calhoon, 1972).

The pressuremeter, cylindrical in shape, is expanded radially against the borehole wall in pressure increments, dilating the borehole. Internal borehole stresses along the central third measurement portion of the probe are assumed to be uniformly radial creating a stress field, corresponding "exactly neither to a condition of plane stress nor plane strain" (Gibson and Anderson, 1961), all the non-uniform, non-cylindrical induced stresses or "end conditions" occurring within the zones of the guard cells (Fig. 3.1). The pressuremeter test may therefore be treated as a linearly elastic, axially symmetrical thick-walled cylinder problem.

A widely recognized theoretical treatment of the load-deformation relationships, as measured by the pressuremeter, is Gibson and Anderson (1961). Their equation for the modulus of deformation as determined from the pseudo-elastic stress range (Fig. 3.4) was used in this thesis. The equation is based on pressure-volume measurements in an elastic, homogeneous and isotropic soil medium.

Palmer (1971) presents a more realistic solution in terms of plastic behavior that is of particular interest to those working in soft soils and high deviator stress situations where strains and non-linearity are pronounced. Apparently Menard attempted an elastic-plastic analysis but did not complete the work (Palmer, 1971).

In drilling the borehole the in situ lateral stress field is altered significantly in the region of the borehole. Prior to advancing the borehole, a uniform compressive lateral stress field can be assumed

to exist. Upon advancing the borehole this stress field is altered with radial and circumferential stress gradients being established in radial symmetry with respect to the borehole axis. The radial stresses are compressive and reduce to zero at the borehole surface. The circumferential stresses are also compressive and increase to twice the original in situ lateral stress value at the borehole surface. Upon reloading the soil with the pressuremeter probe, the stress gradients caused by the creation of the borehole are reduced in magnitude. When the internally applied pressure of the probe reaches the original in situ lateral stress level the radial and circumferential stress gradients will have vanished and the in situ stresses are assumed to have been elastically restored.

Po (Fig. 3.4) corresponds to the stress level at which the in situ lateral stress has been restored in the pressuremeter test. Therefore in principle it is possible to establish Ko at any particular level. However, in practice the stress at which Po occurs is not sufficiently defined to allow an accurate assessment of Ko. Soil disturbance at the face of the borehole and non-cylindrical boreholes may have contributed to the inability of the pressuremeter test to accurately reveal the Po stress level.

The amount of soil significantly stressed by the University of Alberta probe is a cylinder of height 8 inches, inside diameter 3 inches (Nx probe) and outside diameter 30 inches. At a diameter of 30 inches the magnitude of the pressuremeter induced radial and circumferential stresses is 1% of the internal applied stress according to the relationship

$$\sigma_r = -\sigma_\theta = \frac{a^2 P_i}{r^2} \qquad 2.1$$

Equation 2.1 is the elastic stress distribution solution of an infinitely thick walled hollow cylinder subjected to an inner uniform pressure P_i (Timoshenko and Goodier, 1951). This plane stress solution is for an isotropic and homogeneous medium but serves to adequately represent the induced stress field.

The pressuremeter has seen use mainly as an instrument for measuring undrained shear strength and undrained deformation modulus.

It has been the observation of Gibson and Anderson (1961), Meigh and Greenland (1965) and Higgins (1969) that undrained strengths from pressuremeter tests are somewhat greater than CU triaxial strengths at shallow depths, the disparity increasing with depth.

Gibson and Anderson's experience was with overconsolidated London clay in which they used triaxial data from Skempton (1961) for comparison with their pressuremeter strengths.

In regard to the difference in strengths obtained Gibson and Anderson state:

> The reasons for this have not yet been fully explained, but it is known that the shear strength mobilized on a vertical plane - as in the pressuremeter test - is greater than on planes inclined at about 45 deg. to the horizontal - as in the triaxial compression test - due to the higher lateral effective pressure in the ground compared with the effective overburden pressure.

Also offered is the common explanation of sample disturbance.

Meigh and Greenland (1965) made their observations on the Coal Measure mudstone and sandstone and sample disturbance was their explanation for strength differences obtained.

Higgins' experience was with soft cohesive soils and unconfined

compression tests as well as consolidated undrained triaxial tests were performed. As expected, the unconfined strengths were lower than the CU strengths.

It is generally reasoned that the pressuremeter strengths tend to represent the average shear resistance of the soil due to the large volume of soil involved in a given test.

With respect to deformation moduli, Meigh and Greenland (1965) found reasonable correspondence between pressuremeter and plate bearing tests, run on the Keuper Marls, Bunter sandstones and Coal Measures. They suggest that a difference is to be expected, inferring anisotropy when stating "the relationship between the moduli for the two tests is complex".

An increase in E with depth, based on average pressuremeter values, was most pronounced in the silty mudstone Coal Measures on high ground near Wakefield Yorks. The water table lay below the depth of investigation and with the exception of one hole that was water-flush diamond drilled all holes on the high ground were sunk dry using wagon drilling air-flush methods. The wet hole produced comparatively lower values of E. This behavior was also exhibited in tests in Bunter Sandstone at Daresbury, Cheshire.

Calhoon (1969) reports similar results to Meigh and Greenland (1965) for agreement in moduli between pressuremeter and plate loading tests. At the University of Florida at Gainesville, plate loading, pressuremeter and SPT tests were performed in very loose fine sand of about 20% relative density. For an 800 p.s.f. loading the pressuremeter test and SPT tests predicted settlements of 0.15 of an inch and 8 inches respectively. Settlement of 0.20 inches was recorded for the 1 ft^2

circular plate statically loaded to 800 p.s.f. a distance of less than 3 feet from the pressuremeter hole.

According to investigations made by Layne-Western Co., Aurora, Illinois (Calhoon, 1969), pressuremeter settlement predictions agree very well with settlements estimated by the Meyerhof method (Meyerhof, 1965) using SPT values corrected for overburden pressure. The conventional SPT had predicted much higher values for all the sand types involved.

The Layne-Western findings on lacustrine and alluvial clays and silty clay tills showed no appreciable difference between pressuremeter and conventional settlement predictions. The pressuremeter predictions were generally about 30% less on average. The conventional analysis used "laboratory consolidation tests (with reload cycles)" as their data source.

A Layne-Western case of interest is the 33 foot deep by 120 foot diameter excavation in silty-clay till which had a 3 day bottom heave of 0.3 inches and a pressuremeter prediction also of 0.3 inches.

Dixon and Jones (1968) write of the credence given to pressuremeter data over static and dynamic laboratory testing and seismic measurements for the design of the Castaic Surge Chamber and Stokes Canyon Tunnel in California. Pressuremeter values were chosen because they better represented the rock mass. Laboratory tests were necessarily run on higher than average quality samples and gave upper bound moduli. Seismic measurements served best as a check on the trend of pressuremeter and laboratory values gained throughout the region of concern.

Hendron et al. (1970), from pressuremeter and laboratory UU

tests on overconsolidated shale, obtained data that would strongly indicate the possibility of relating initial water content to modulus of deformation. Such a relationship would be significant in that it would allow the use of initial water content as an index property which could be used to estimate the deformation modulus of a given soil at locations where test data are not attainable.

They observed further that the undrained moduli obtained from the pressuremeter tests were 3 to 7 times greater than those determined in the laboratory.

2.4 Summary

Until recently, the treatment of elastic settlement of geotechnical structures has been grossly inadequate, both in analytical technique and measurement of elastic soil parameters. The analytical procedures have developed rapidly from the early Boussinesq approach through to the powerful finite element method of today. In contrast, improvement of the measurement of the modulus of deformation has not progressed at a comparable rate.

The finite element technique makes possible the realistic treatment of nonlinear, anisotropic and non-homogeneous soil conditions under practically any stress and displacement boundary condition.

Elastic parameter evaluations have characteristically been derived from in situ or laboratory tests under static or dynamic loading. Test selection is usually dictated by the economics of the project and the anticipated field loading conditions.

Static undrained modulus tests on laboratory samples have traditionally been the unconfined compression and consolidated undrained

triaxial tests. The laboratory values are invariably less than the field moduli, often to the extent of an order of magnitude. The values, derived from these tests have been shown to be sensitive mainly to sample disturbance, shear stress level, confining pressure and amount of secondary consolidation allowed before testing in undrained shear. Sample disturbance is undisputably the major contributor to low laboratory E values.

Of the static in situ tests, the pressuremeter is gaining in popularity as a reliable instrument for measuring modulus in sand, cohesive soils and soft rock. The pressuremeter also gives undrained strengths that are representative of the soil mass and these values tend to be somewhat higher than undrained triaxial strengths for intact materials, the difference increasing with depth in the deposit.

There is an unfortunate lack of published case histories giving performance of structures that were constructed using the pressuremeter as a design tool.

CHAPTER III

THE PRESSURE PROBE AND THE IN SITU PRESSURE TEST

3.1 Introduction

The in situ pressure probe, though relatively simple in concept and design, is like most other test instruments, in requiring continued development as new test situations are encountered. Most improvements come as a result of experience with the existing model, based on an understanding of what conditions must be satisfied in the test and how the instrument meets these requirements. Therefore it is important to have a thorough understanding of the mechanical systems of the pressuremeter along with an awareness of their efficiency. Experience is often the means to this awareness.

Some necessary modifications were installed on the University of Alberta pressuremeter for the CN and AGT towers investigation. Further design improvement is necessary for pressuremeter testing in soft soils.

Testing procedure is equally important if the best possible data are to be obtained and proper correction factors must be recognized and applied for meaningful reduction of this data.

3.2 The Pressuremeter

The pressuremeter system consists of an expandable probe and a pressure volumeter. Schematic representation of the system is presented in Fig. 3.1.

The probe consists of a hollow aluminum shaft with a spring loaded lower section. This shaft is surrounded by two flexible rubber

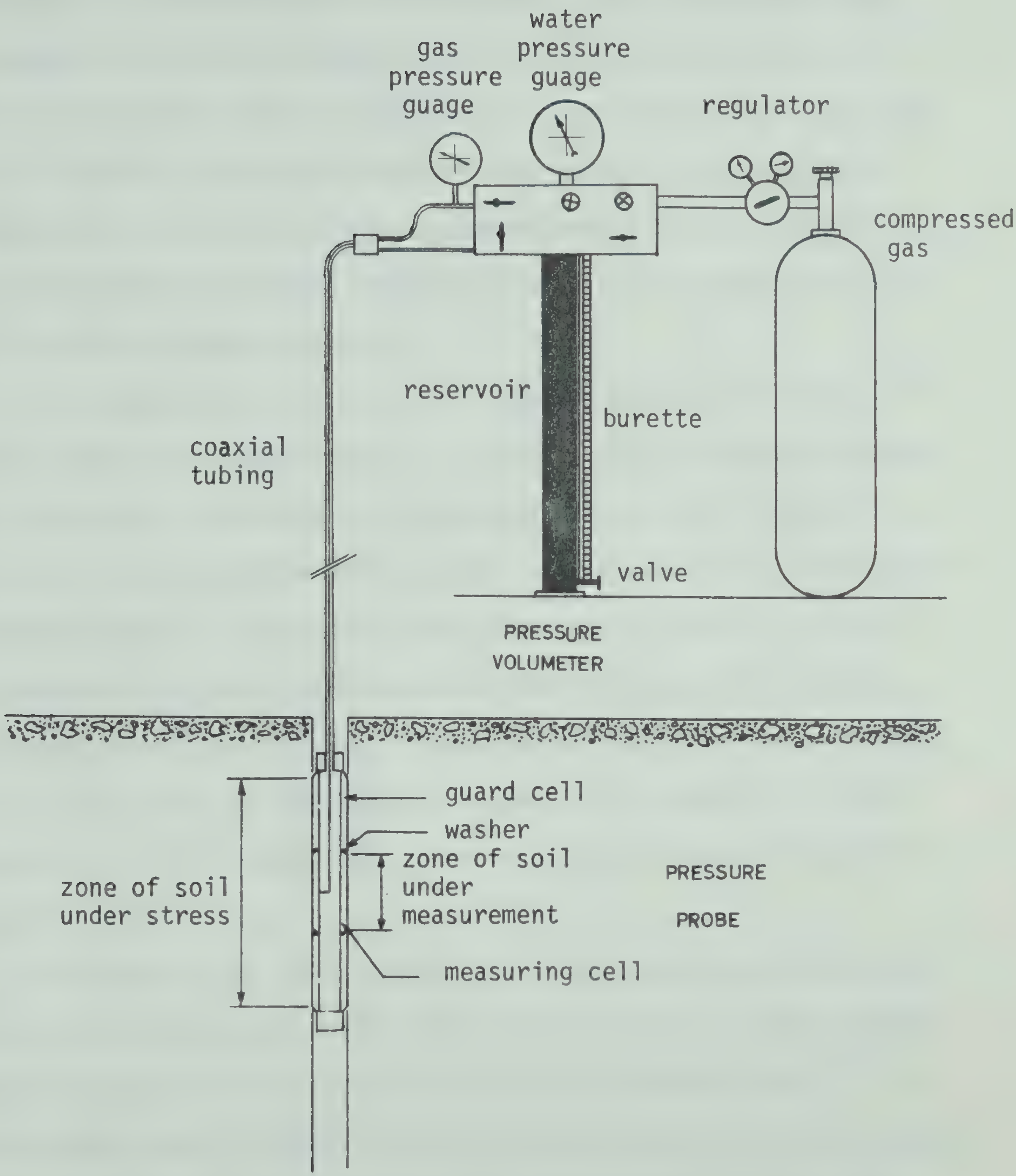

FIG. 3.1 SCHEMATIC DRAWING OF PRESSUREMETER EQUIPMENT

membranes. An 8 inch rubber membrane made from a motorcycle inner-tube makes up the inner membrane and is located centrally along the shaft where it is secured at both ends to form the interior measuring cell. A 29 inch rubber hose similar to a radiator hose, except for longitudinal reinforcing, forms the outer membrane and is clamped at both ends forming air-tight exterior guard cells and protective cover for the interior measuring cell.

The construction of this outer membrane is very important to the performance of the pressuremeter. It must be able to expand radially but not axially. The need to expand axially upon radial expansion is satisfied by a shortening of the probe. The lower spring loaded section is pulled upward by the induced axial force upon expansion. The re-inforcing cord must run longitudinally and no bias can be tolerated. The longitudinal reinforcing is necessary to prevent friction folding of the outer membrane resulting in wedging of the probe in a tight borehole and also longitudinal expansion past the ends of the probe if the hole is too large.

The guard cells are activated by gas pressure and the measuring cell is pressurized with water and kept at a slightly higher pressure than the guard cells to insure contact with the borehole wall. The guard cells serve to create near plane strain conditions in the region of the measuring cell by increasing the axial extent of the radial stress field (Gibson and Anderson, 1961).

The incompressibility of water makes it an ideal medium for the measurement of pressure-volume behavior.

The probe is connected to the pressure-volumeter by coaxial tubing, the inner tube carrying the water phase. Apart from being more compact,

coaxial arrangement provides a means of preventing expansion of the water line upon pressure application.

The pressure-volumeter consists of a water supply tank, burette and a system of valves for control of pressure on the air and water phases. The pressure on the water phase, at the probe elevation during a test, is equal to the applied pressure plus the difference in head between the probe and pressure-volumeter (under static flow conditions). The excess pressure required in the measuring cell is about 10 p.s.i. and therefore testing at depths greater than about 25 feet requires that less pressure be applied to the water phase than to the gas phase. The required pressure difference increases with depth at the rate of 0.434 p.s.i. per foot which is the increase in excess static head. This differential pressure requirement is difficult to maintain through the use of separate regulators and therefore a variable pressure reduction valve was built to solve this problem. Fig. 3.2 shows the pressure control system used in which pressure reduction valves were used to regulate differential pressure for the water and gas phases from a common pressure source. (Fig. 3.2 was constructed for use in conjunction with section 3.3 in which a full understanding of the figure is developed.)

In order to make the pressuremeter operational it was also necessary to restrict axial expansion of the inner membrane. This was achieved quite simply by placing steel washers at the ends of the measuring cell (Fig. 3.1). The washer diameter was slightly greater than the inside diameter of the outer membrane and were slotted to allow gas to pass through from the guard cells to force water back up to the reservoir upon completion of the test. These washers were of

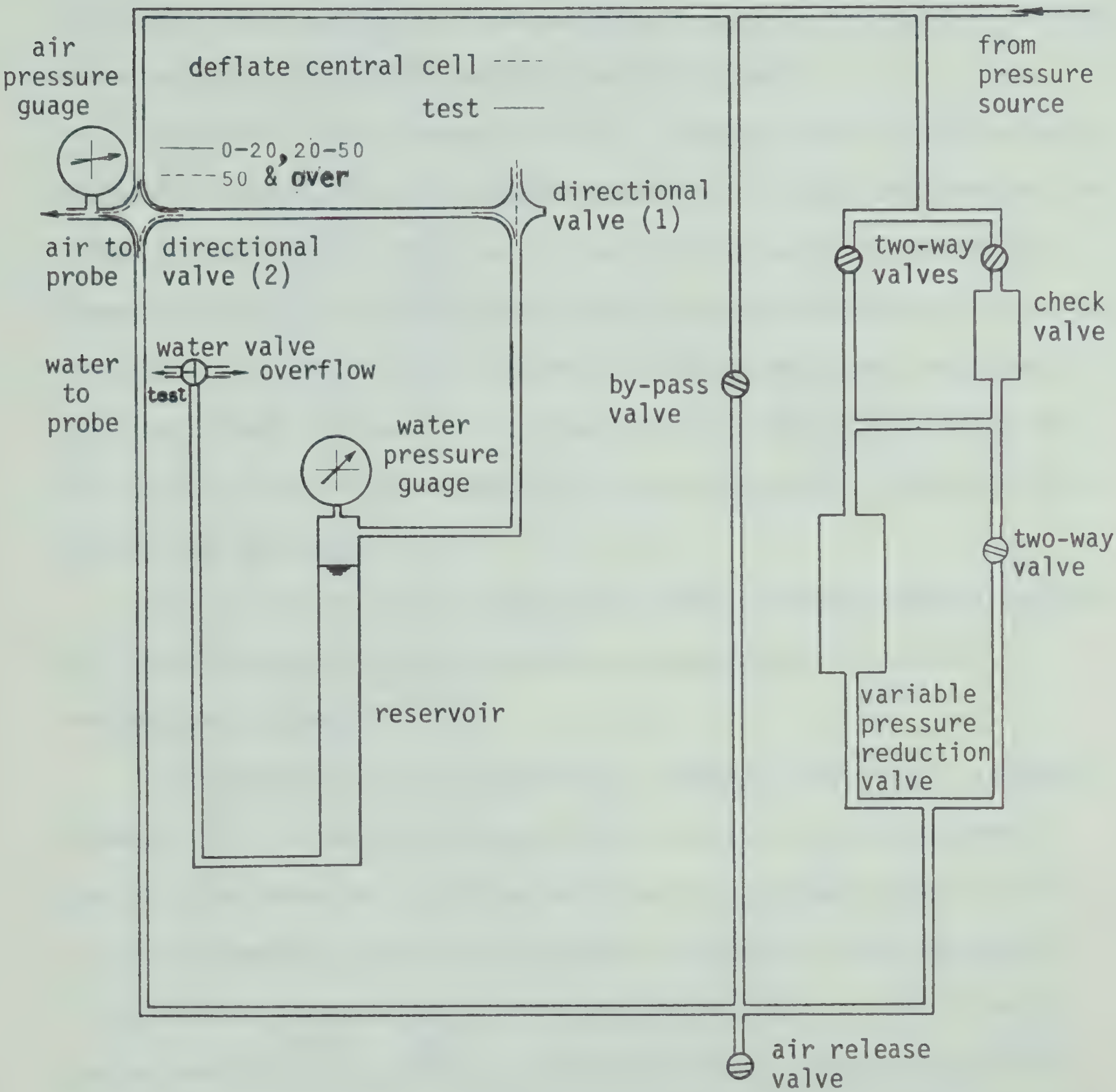

FIG. 3.2 PRESSURE SYSTEM NETWORK OF THE PRESSUREMETER

fixed diameter and operated effectively only in stiff soils and rock. The ideal condition is for these washers to expand, remaining in contact with the outer membrane throughout the entire test.

The probe, as it currently exists, requires water in the borehole to prevent expansion of the measuring cell due to head developed as the probe is lowered into the hole. To shut off the water supply to the probe results in cavitation in the upper low pressure region of the line. Because of the propensity of the soil to soften and scour during wet drilling, there is a need for a probe that will function in a dry hole. The design of such a probe amounts to the devising of a method of flow control at the probe.

Fig. 3.3 is a design suggestion to meet this need and the need for a 4 inch[1] diameter probe with variable diameter axial expansion restrictors (washers).

The operation of the flow control system of Fig. 3.3 is straight forward. The valves allow flow in one direction only as indicated. The variable pressure control valve is set to a predetermined cracking pressure somewhat greater than the head developed at test elevation. This cracking pressure is set from calibration marks on the valve or can be set directly by: fully loading the valve; applying the desired cracking pressure to the water phase; and then unloading the valve until the burette indicates passage of water through the valve.

The success of this design depends upon the ability of the valve to transfer water across a high pressure gradient without excessive cavitation occurring.

[1]Drilling companies in the Edmonton area are better equipped to produce 4-inch diameter boreholes thereby establishing an economical need for a 4-inch diameter probe.

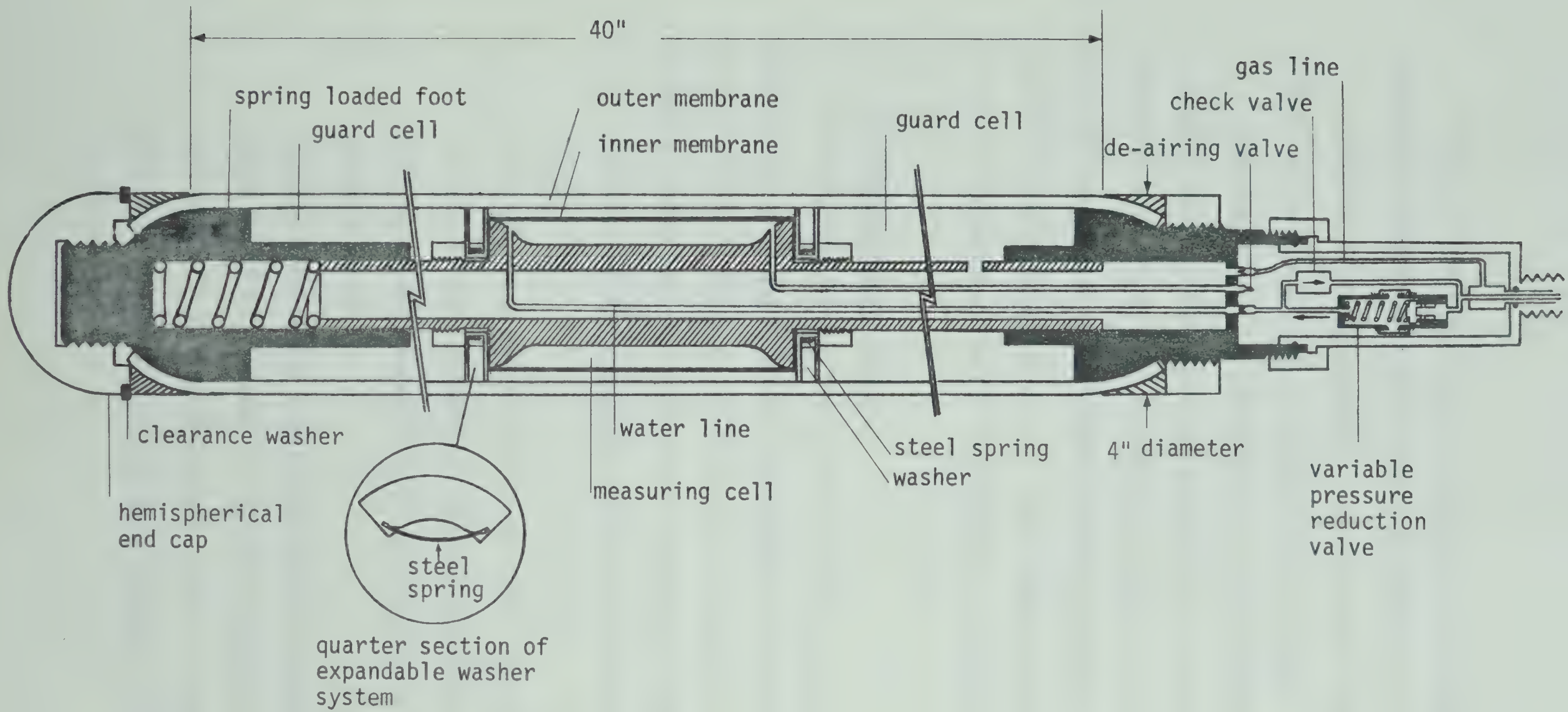

FIG. 3.3 SUGGESTED PROBE DESIGN IMPROVEMENTS

Water is forced out of the measuring cell through the check valve. A cracking pressure of 1 or 2 p.s.i. is desirable for this valve.

Like the valve arrangement, the expandable washers described in Fig. 3.3 serve only as a suggestion for a design requirement.

With respect to the design of a 4 inch diameter probe it should be remembered that the axial force applied to the outer membrane increases as the square of the diameter whereas the perimeter along which the resistance is developed increases directly as the diameter. With this in mind in going to a 4 inch diameter probe a new method of end restraint may have to be instituted for the outer membrane.

3.3 The Pressuremeter Test

Prior to performing the first borehole pressuremeter test, there are 24 preparatory steps that must be carried out. Variations to this procedure are possible but the end result is the same. The steps are listed below with pertinent commentary where necessary.

1. Ensure that the regulator will handle the expected pressure range.
2. Fill the water reservoir to the top of the graduated ruler. The use of food coloring in the water facilitates easy reading of the manometer and location of air bubbles in the water line.
3. Connect probe water inlet tube to the volumeter. At this stage there is no water in the line leading to the probe but when there is and when possible, this line should be disconnected under about 5 p.s.i. line pressure. This serves to

prevent air from entering the line at the moment of separation and at the instant of contact in the case of reconnecting the line.

4. Close by-pass and air release valves.
5. Connect gas bottle to volumeter first ensuring that both regulator and bottle are shut off.
6. Place directional valve (1) on "Test". This exposes the water phase to the pressure source.
7. Place directional valve (2) on "0-20". This connects the water phase directly to the pressure source and the gas phase indirectly through the reducing valves.
8. Place the water valve on "Test". This opens the line from the reservoir to the probe.
9. Apply 20 p.s.i. through the regulator. When the line has filled turn water valve to "Closed".
10. Connect coaxial line to probe. The inner water line should extend about 1 foot past the end of the outer air line under zero axial load on the lines. Therefore it is necessary to stretch the outer line in order to join the coaxial tube to the probe. The greater length of the inner line ensures that it does not get pulled out of the probe due to axial load on the outer line during raising and lowering of the probe in the borehole. Two people are generally required to perform this operation but for this de-airing process it is sufficient to connect only the water line.
11. Turn water valve to "Test". Water will thus be injected into the measuring cell.

12. Turn water valve to "Closed" when a significant bulge has appeared on the central part of the probe.
13. Disconnect water line from probe and allow water to be ejected from the probe thus expelling any air bubbles from the measuring cell.
14. Repeat operations 10 to 13 several times then connect both lines of the coaxial tubing.
15. Shut off pressure at regulator.
16. Open air release valve to depressurize the system and place directional valve (1) on "deflate central cell".
17. Disconnect probe water inlet tube from volumeter and refill the reservoir.
18. Connect both water and air tubes of probe to the volumeter.
19. Place probe inside Bx casing (or Nx if using Nx probe).
20. Close air release valve.
21. Place directional valve (1) and water valve on "Test".
22. Apply 40 - 50 p.s.i.
23. When volume reading is steady, bleed water off by turning water valve to "bleed" until a reading of 225 is indicated (or 462 when using an Nx probe. In this case do not omit changing the graduated ruler). Steps 19 to 23 serve to initialize the volume reading of the manometer so that volume may be read directly off the graduated ruler.
24. Depressurize system as per step 16.

The probe and pressure-volumeter are now ready for testing.

The borehole becomes the next concern and the manner in which it is made is very important.

The ideal borehole is one in which the axis is straight, the diameter is constant and the soil is undisturbed.

At borehole CH1 the 3 inch diameter hole was advanced rapidly using an auger. The soil was overconsolidated till, the hole was dry and the pressuremeter time-volume curves were definitive.

Borehole CN1 was advanced as rapidly as possible using a 3 inch diameter wing bit. Wet drilling procedure was used and the soil was till, sand and shale. The sand and shale gave good volume-time curves that improved with depth but the curves for the till were inferior. This may have been due to the method of drilling. The till contained much sand and gravel, impeding the rate of drilling thus allowing greater opportunity for scouring to occur.

Borehole AGT1 was advanced through shale using a 3 inch diameter wing bit and gave very good volume-time curves that improved greatly with depth.

It would appear that in a uniform material like sand and shale, wet drilling at a high rate of advancement produces a good pressuremeter borehole. Experience in till would suggest that dry augering produces a superior hole in this soil.

Fast rate of borehole advancement may tend to cause greater soil disturbance but serves to produce a geometrically superior hole by minimizing scour and vibration effects.

When the borehole has been drilled the next item of importance is the pressuremeter test. If tests are to be made to depths greater than about 50 feet, the borehole must be filled with water. All the necessary steps involved in a typical pressuremeter test are outlined below in point form with details where applicable.

1. Set differential pressure

   a) For depths 0 - 20 feet

      - place directional valve (2) on "0.- 20"
      - close by-pass valve
      - open check valve

   b) For depths 20 - 50 feet

      - place directional valve (2) on "0 - 20"
      - open by-pass valve

   c) For depths greater than 50 feet

      - place directional valve (2) on "50 & over"
      - close by-pass valve
      - close check valve
      - load the pressure reduction valve (i.e. fully tighten the valve)
      - disconnect the air line
      - place water valve on closed
      - place directional valve (1) on "Test"
      - pressurize the system through the regulator to a pressure greater than the desired pressure differential. Only the gas phase will pressure up with the pressure level being indicated on the gas phase pressure gauge.
      - unload the pressure reduction valve until the water pressure gauge indicates the proper reduced pressure valve
      - shut off regulator
      - open air release valve

- reconnect air line

2. Close air release valve
3. Place water valve on "Test"
4. Grease probe - this is necessary in order to keep the outer membrane from pulling out at the lower end when inserting the probe into the hole. Although the wing bit diameter was 1/16 inch greater than the probe it was nearly always necessary to force the probe into the hole. Often several hundred pounds force was necessary and if the probe was not greased problems developed.
5. Lower probe to desired elevation - the drill rig is used for this operation with "A" rod used to lower the probe. It is important to keep the coaxial line taut to prevent tangling and line damage.
6. Open regulator until the gas phase pressure is approximately 15 p.s.i. greater than the hydrostatic head at test elevation. The water pressure will track at the preset pressure differential. This excess pressure is necessary for seating the probe.
7. Allow several minutes for the probe to establish equilibrium.
8. Close water valve and note initial pressure and volume readings.
9. Apply pressure increment through regulator.
10. Turn water valve to "Test" starting the clock simultaneously.
11. Take time-volume readings until change in volume with respect to time becomes constant. Typical examples are contained in Appendix A. When volume changes resulting from pressure increments are too small to be read adequately, the sensitivity

of the instrument can be increased 64 times by closing the tap at the bottom of the sight-glass. In this case only the liquid within the sight-glass is injected into the probe. Each millimeter on the scale ruler represents 1/32 of a cubic centimeter as against 2 cubic centimeters when the top is open.

12. Repeat operations 8 to 11 about 6 times over the predetermined stress range or until volume change becomes excessive. Plot at least the first two time-volume curves of the test to establish a reasonable amount of time to allow for each pressure increment. Large pressure increments should be avoided in softer material if a well defined pseudo-elastic range is to be obtained. About 1 hour per test can be anticipated at first. Once the behavior of the soil has been established four pressure increments at the proper stress level are all that are required for the determination of modulus of deforamtion. This will decrease the time to 35 minutes per test.
13. Close off pressure from regulator.
14. If an unload curve is desired, decrease system pressure in increments using the air release valve, taking readings according to steps 8, 10 and 11.
15. Lower system pressure by opening the air release valve to a value lower than the failure load.
16. Set directional valve (1) to "deflate central cell".
17. When all the water has returned open the release valve to depressurize the system.

18. Remove probe from the advanced 3 inch diameter hole at a slow rate keeping the coaxial line taut.

The test now completed the hole is advanced to the next test elevation.

It would be desirable to perform several if not all the pressuremeter tests required in a given borehole in one sequence of tests. From the detailed time sheets maintained by Artesia Drilling Ltd. a saving of about 33% (per 100 ft. depth with 10 ft. testing intervals) with respect to total time and about 60% with respect to drill time could be realized by first drilling the entire hole and then performing the entire sequence of pressuremeter tests. The saving in drill time is based on the supposition that the drill rig is employed only to drill the hole and not to handle the probe and that the hole will remain open. The probe could be handled by hand if a larger hole could be tolerated by the probe. A probe with expandable washers would satisfy this requirement.

A model attempt to do this by advancing the 3 inch hole 20 feet resulted in a borehole that was too large for the first 6 feet and too small for the last 6 feet. As you advance a borehole using wet drilling procedures the upper part of the hole is subjected to vibration effects of the drill stem and longer periods of scour. Increasing the rate of drilling excessively with depth, to reduce scour time in the upper region of the hole, results in greater vibration effects and a hole that is too small at the bottom. A solution might be to use air drilling and a probe similar to that described in Fig. 3.3. The present probe is too sensitive to borehole diameter but expandable washers should decrease this sensitivity.

3.4 Data Analysis

In the pressuremeter test the modulus of deformation is determined from the relationship between the pressure increments and the corresponding volume change measurements. Fig 3.4 shows a complete volume-pressure curve for the pressuremeter test. It is the typical representation of soil behavior under the influence of the cylindrical stress field of the probe.

A value of Ko is theoretically derivable from the "lead-in" phase if this portion of the curve reflects only the restoration of lateral in situ stresses. Due to disturbance effects and water softening at the borehole face, the point where the "lead-in" phase ends and the pseudo-elastic phase begins is not definite, thereby introducing uncertainty into Ko evaluations.

An approach to the pressuremeter test from the point of view of determination of undrained strength is given by Menard (1964) in which a formula is given for the determination of undrained strength based on the plastic limit - Pu (Fig. 3.4).

Only the pseudo-elastic linear portion of the curve was of importance to the problems of this thesis and therefore the tests were terminated within this pressure range.

The formula for deformation modulus developed by Gibson and Anderson (1961) was used in this thesis in the form:

$$E = \frac{2Vo\ (1 - \mu)\,\Delta P}{\Delta V}\ , \qquad 3.1$$

where Vo = vol. at measuring cell at the beginning of the pseudo-elastic phase,

μ = Poisson's ratio,

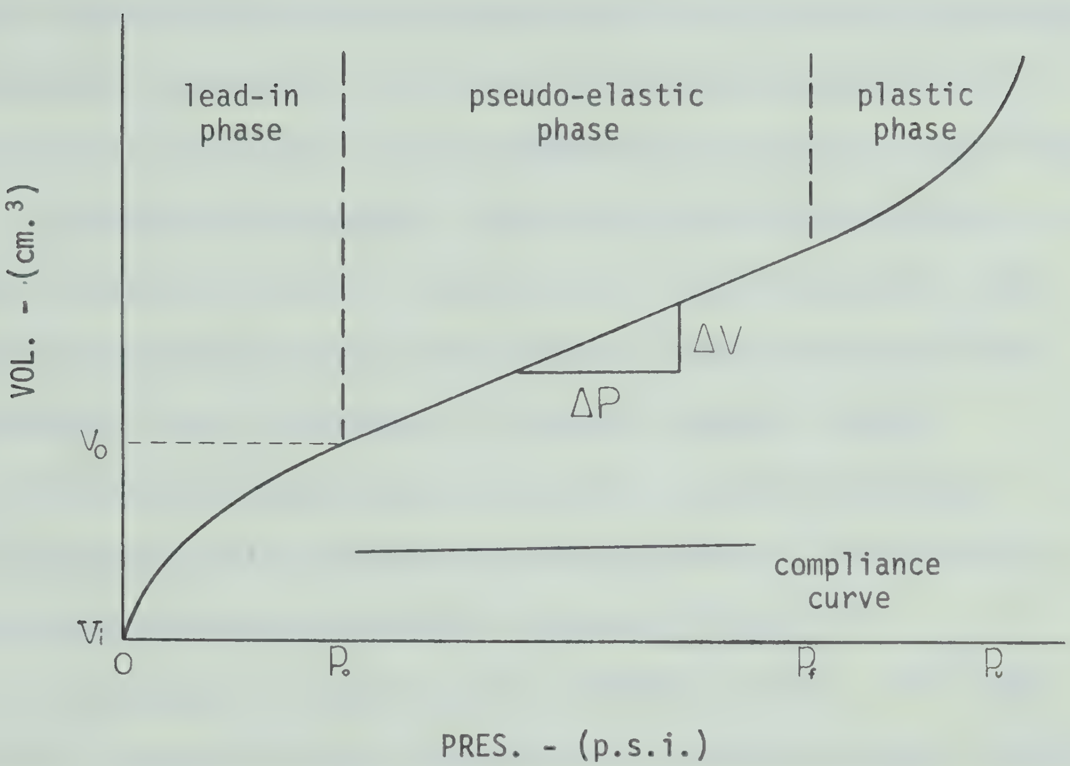

FIG. 3.4 COMPLETE VOLUME – PRESSURE CURVE OF THE PRESSUREMETER TEST

and $\Delta P/\Delta V$ = slope of curve in pseudo-elastic phase where ΔV is the corrected volume change corresponding to ΔP.

Pressure reduction at the probe due to friction loss in the water line under flow conditions, plus the amount of pressure required to increase the measuring cell by ΔV due to the resistance of the probe alone, comprise the correction factors that can be applied to the pressure increment ΔP in equation 3.1. These corrections were very small for the flow rates and volume changes encountered and were thus neglected.

Volume change measured is the sum of the volume increases due to: elastic response of the soil; compliance of the pressuremeter; and apparent compressibility based on imperfect probe contact with the borehole wall (also referred to as surface roughness effect).

The volume change due to the elastic response of the soil is the desired quantity and is determined from the measured volume change by subtracting compliance and surface roughness effects.

Compliance is a measure of the stiffness of the system and includes: compressibility of the membranes which tend to be a function of the pressure and volume of the measuring cell; compressibility of the water; and the expansion of the water lines and reservoir. The error in volume measurement due to compliance is measured by performing a pressuremeter test inside a steel casing. Pressure-volume readings are taken from which the volume change due to compliance for any pressure increment at any pressure level can be determined. Tests inside casings of different diameters showed no detectable volume dependence. The pressure-volume curve for complaince is illustrated schematically in Fig. 3.4 and in respect to the slope of the average pressuremeter test is observed to be of negligible contribution. Compliance error of course

in importance with soil stiffness.

Error due to surface roughness was assumed to be eliminated in the "seating-in" phase.

A typical volume-time curve (corresponding to a given pressure increment) from which ΔV and Te, the elastic time interval, is obtained is given in Fig. 3.5. The method of determining Te and consequently ΔV as indicated in Fig. 3.5 was chosen with the idea that the initial linear portion of this curve represented the elastic soil response. It is interesting to note that this method of producing tangent lines as opposed to arbitrary selection of Te and ΔV gave the best linearity in the pseudo-elastic stress range of the volume-pressure curves for pressuremeter tests in which both ΔV selection techniques were used.

Ideally volume change with time should have decreased to zero upon completion of the elastic response. There was a tendency for this to occur with depth as the shale became stiffer and the permeability decreased. Figs. 4.3 and 4.4 show the relative pronouncement of this "creep-like" phenomenon as a function of soil type and depth. It is the author's opinion that consolidation effects, plastic movement of the softened soil at the borehole surface, and variable diameter of the borehole due to sluffing are the main contributors to this behavior.

Prior to field use of the pressuremeter, a calibration test was performed to assist in the interpretation of the field results.

A sand-cement mixture consisting of 8% cement, 10% water and 82% sand by weight was prepared from which a thick-walled cylinder and 5 six-inch standard concrete cylinder specimens were cast. The thick-walled cylinder was 3 feet high with inside diameter of 3 inches and outside diameter of 30 inches. Density control was maintained by a

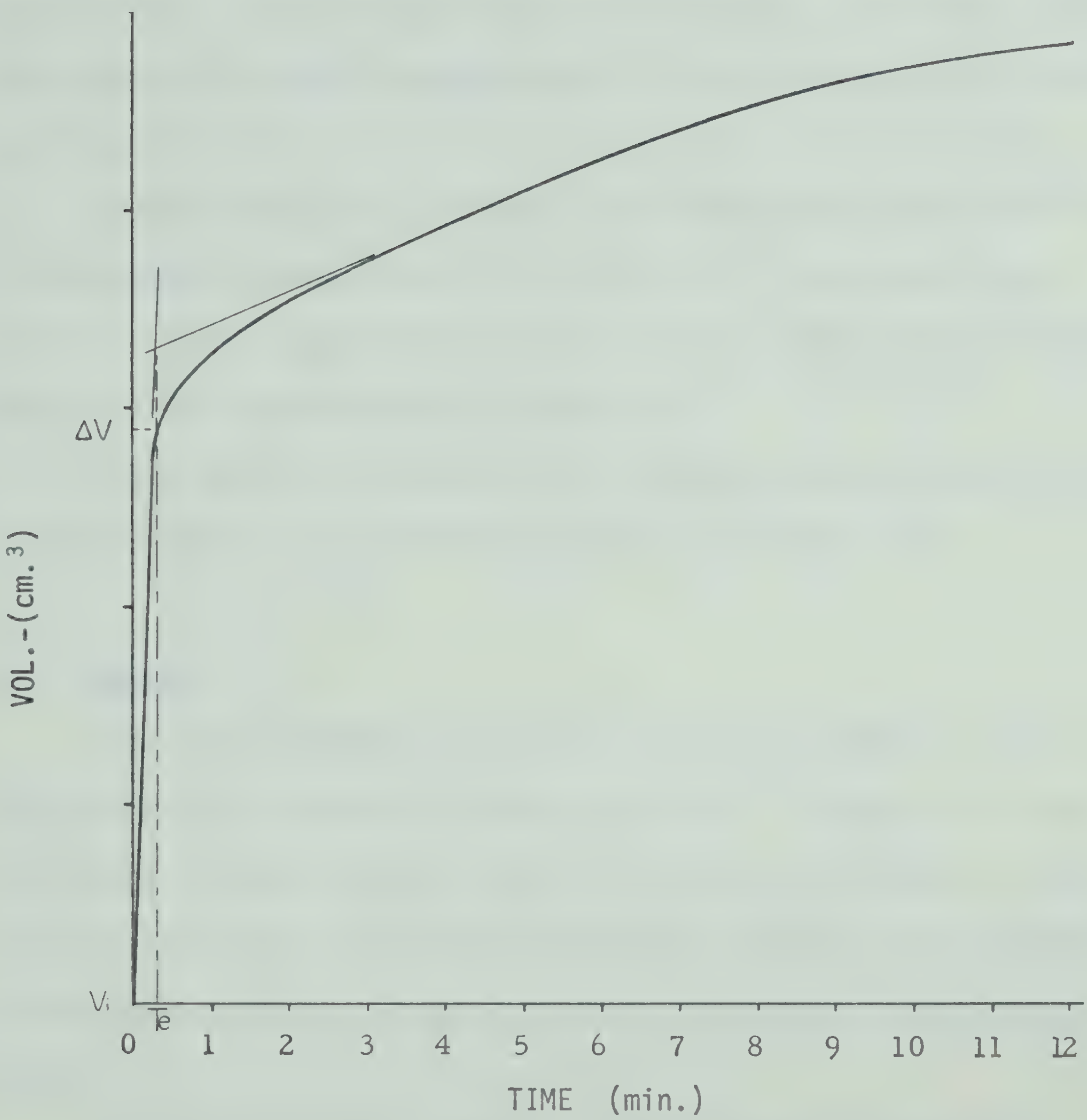

FIG. 3.5 TYPICAL VOLUME - TIME CURVE

fixed number of blows per area and the mixture was applied in 3 inch layers. The blows were administered by hand using a 2 x 4 with the intensity and distribution controlled by the operator's judgment. Density tests, using a mercury immersion technique, were later performed revealing a 3% greater average density of the cylinder samples than the test sample.

Taking Poisson's ratio equal to 0.3 the pressuremeter test gave a deformation modulus value of 548,000 p.s.i. The average secant modulus from the unconfined compression tests at 40% ultimate load for the 5 cylinder specimens was 555,000 p.s.i.

This result is reassuring but of course cannot be used as conclusive evidence of the accuracy of the pressuremeter test.

3.5 Summary

The pressuremeter probe in its present form requires water in the borehole for testing below about 25 feet. The probe also cannot tolerate a borehole diameter greater than about 3.25 inches. These requirements result in drilling and testing procedures that are from 33% to 60% more expensive than what could be realized by a more versatile probe.

A possible design solution to this problem is suggested. The main components of the design charge are: expandable end washers for the measuring cell which would allow greater tolerance to variations in borehole diameter; a system of water control valves stationed within the probe to permit dry hole testing; and a 4 inch diameter dimension to accommodate existing drilling facilities in the Edmonton area.

The actual operation of the pressuremeter is relatively simple but common errors can be made by the novice and therefore strict adherence to the outlined procedure should be maintained for the first few tests.

It is important to plot the test results immediately after they are obtained to serve as a guide for subsequent tests with respect to pressure range required and most of all to the degree to which the test is measuring the intended parameter.

Analysis of pressuremeter data, from the point of deformation modulus determination, amounts to selecting ΔV values from the volume-time curves and applying correction factors when applicable; selecting a Poisson's ratio; and solving equation 3.1.

CHAPTER IV

FOUNDATION INVESTIGATION AT THE CANADIAN NATIONAL (CN) AND THE ALBERTA GOVERNMENT TELEPHONES (AGT) TOWERS

4.1 Introduction

In order to determine the suitability of the pressuremeter for predicting elastic deformations it is desirable to have well monitored field cases against which one can compare results.

DeJong (1971) in the development of his Ph. D. thesis had collected a large amount of valuable data on the load settlement behavior of the CN, AGT, Avord Arms and Oxford Towers. Settlement at the CN tower and heave in the AGT excavation were selected for pressuremeter studies. The selection was based upon completeness of necessary data and the relative ease of the required analyses.

Figs. 4.1 and 4.2 give the locations of the CN tower and AGT excavation in the downtown Edmonton area. Also shown are the locations of the respective boreholes used in the foundation investigation.

The highly overconsolidated nature of the bedrock in the area is well adapted to pressuremeter prediction of foundation behavior since the deformation encountered is composed largely of immediate response to loading.

An understanding of the geology of the foundation is desirable in any geotechnical investigation and so a brief account has been given.

The rest of the chapter deals with the pressuremeter and laboratory testing results of the investigation.

4.2 Geology

Four distinct geological deposits constitute the zone of concern with respect to elastic foundation behavior of structures constructed in

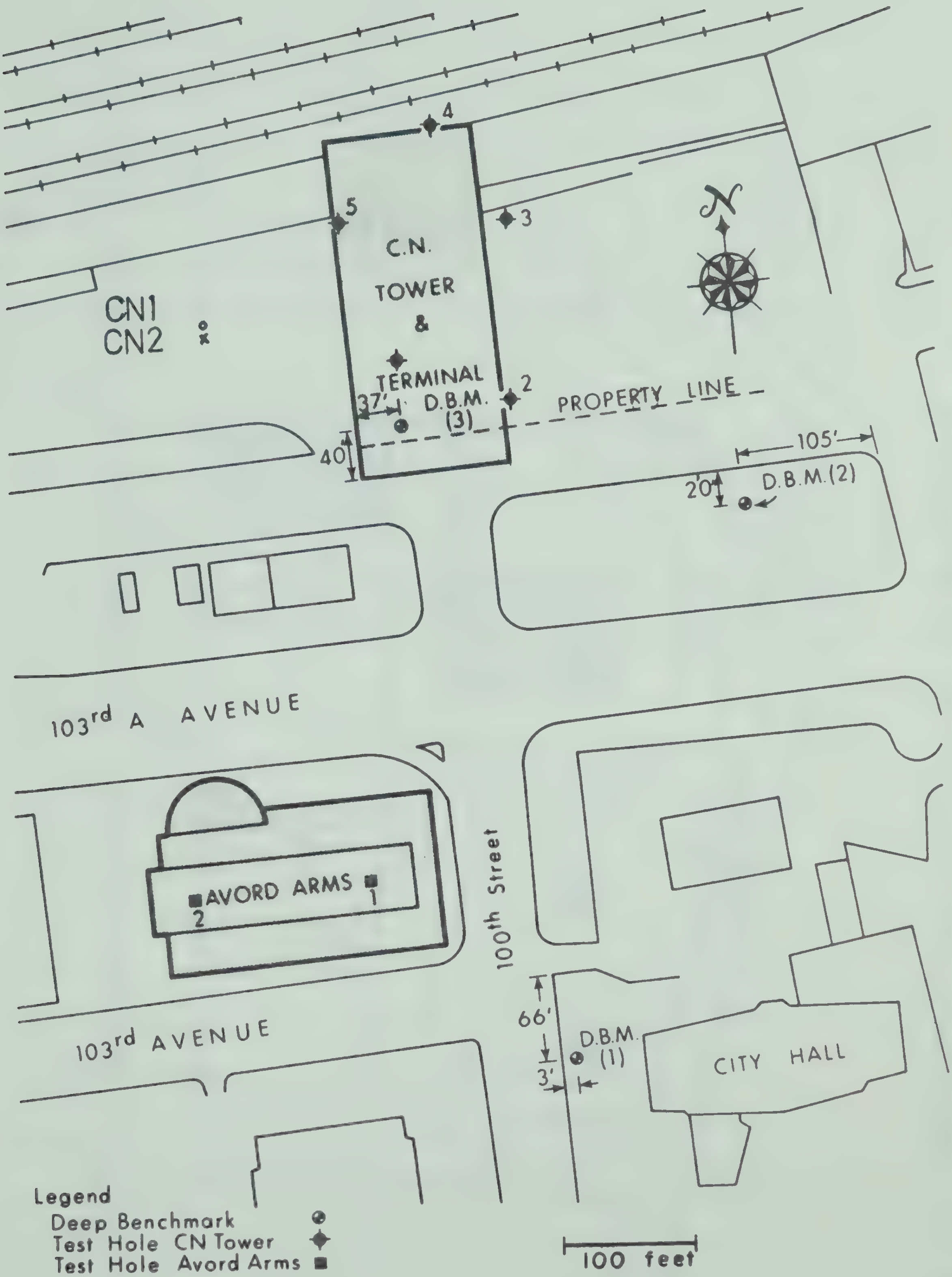

FIG. 4.1 LOCATION OF CN TOWER, PREVIOUS TESTHOLES, AND BOREHOLES CN1 AND CN2 (AFTER DEJONG, 1971)

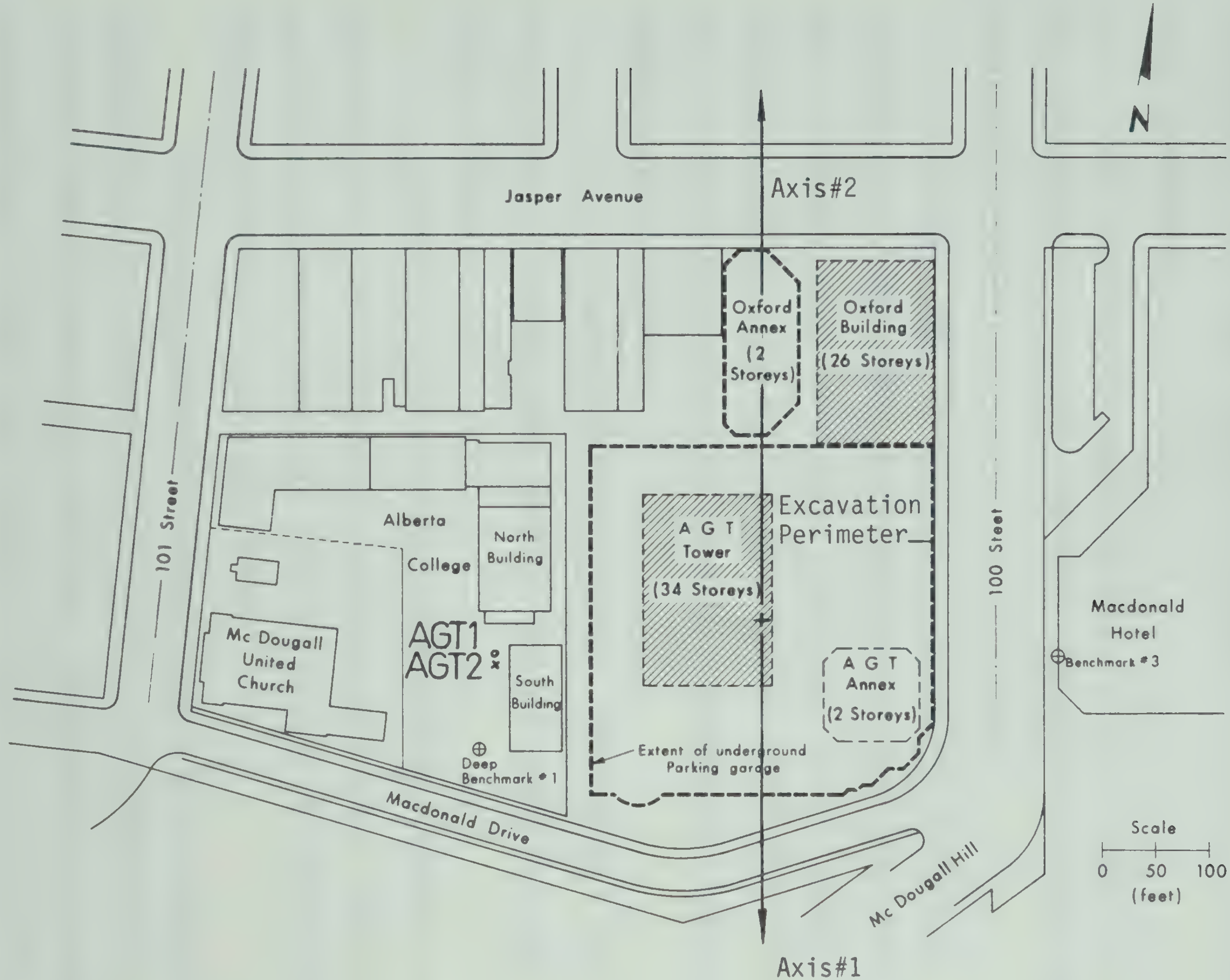

FIG. 4.2 LOCATION OF AGT TOWER EXCAVATION, AXES FOR TWO-DIMENSIONAL ANALYSES AND BOREHOLES AGT1 AND AGT2 (AFTER DEJONG, 1971)

the downtown Edmonton area. The deposits in descending order of deposition and ascending order of importance in their contribution to the elastic deformations in the cases studied are: lacustrine clay; till; sand and gravel; and bedrock.

The lacustrine clay varies in thickness but is about 16 to 20 feet deep increasing in silt content with depth, typical of proglacial lake deposits.

Westgate (1969) reports the existence of two distinct till sheets of Wisconsin age, with pebble orientation being the main identifying feature. Both till sheets, though separated by a thin sand deposit, contain a wide range of stone sizes, coal and iron deposits, and sand lense inclusions all in a sandy-silty clay matrix. The till is extensively fissured and on average about 30 feet deep.

The sands are medium to fine grained, deposited as a result of erosion cycles. They are dense deposits ranging in depth from about 3 to at least 40 feet.

The bedrock is the most important foundation component from the viewpoint of this thesis in that it constitutes the deepest deposit and contains the deformation properties that determine to the largest degree the final magnitude of elastic deformation.

The bedrock, known as the Edmonton formation, is composed of interbedded bentonitic mudstones (shales), and silty sandstones with occasional seams of chert and lignite coal. The shales are a laminated sediment composed primarily of clay-size particles, with laminations that are inclined at about 8° to the horizontal at the AGT and CN Tower sites. The Edmonton formation was formed in a non-marine environment during the Upper Cretaceous Period and is in the order of 1600 feet deep in the Edmonton

area (Ower, 1958).

The source of much of the non-precipitated material brought into the marine basins throughout most of the Upper Cretaceous Period is believed to have been the rising Rocky Mountain region. The location of the western shoreline with the rising land area fluctuated widely throughout this period, as a reflection of crustal movement. This condition led to the intertonguing of the predominantly argillaceous marine and arenaceous or clastic non-marine sediments, common to the Alberta and Saskatchewan sequences. Swamp, lake and lagoonal conditions were frequently developed along the unstable shoreline, resulting in organic rich bands and coal seams within the non-marine deposits.

Due to volcanic activity in the source area (Rocky Mountains), the depositional basins received much volcanic material in addition to the normal weathering products of sedimentary rocks. The main vehicle of deposition of the volcanic debris was the normal erosional-transportational weathering cycle but occasionally there were deposits made directly by air that resulted in distinct bentonite layers. With less rapid arrival of volcanic dust or in regions of more active normal sedimentation, the bentonite occurs in varying concentration, diffused throughout the shale.

These sediments were subsequently subjected to loads of from 1200 to 2500 feet of overburden before the Pleistocene Age of glacial activity. As a result the shales are presently in a heavily overconsolidated state and Ko values greater than 1 can be expected to exist away from river valleys.

Upon erosion the high in situ lateral stresses may have been relieved near the surface once erosion had progressed sufficiently to cause

passive failure, however there is no evidence that this has occurred. Some lateral stress relief in the upper 200 feet of the Edmonton formation in the downtown area of Edmonton however will have occurred as a result of downcutting of the North Saskatchewan River.

4.3 Pressuremeter Testing at the CN and AGT Towers

Boreholes CN2 and AGT2 (Figs. 4.1 and 4.2) served as a reconnaissance of the zone of concern, as well as providing samples. The pressuremeter explorations were based upon the stratigraphic information from these boreholes.

Pressuremeter investigation of the CN and AGT Tower foundations was carried out in boreholes CN1 and AGT1 respectively. The pressuremeter tests in till at the CN site were substituted for by pressuremeter tests made in the till at borehole CH1 due to greater confidence in results.

Figs. 4.1 and 4.2 show the location of CN1 and AGT1, in relation to the respective foundations they represent. CH1 is located at 100 St. - 102A Ave. about 1000 feet due south of CN1.

CN1 and AGT1 were drilled by Artesia Drilling Ltd. of Edmonton using a Failing 1500 rig. The rig was capable of dry drilling using air circulation but the probe required water in the borehole in order to function so water circulation was used.

To achieve the borehole specifications described in Chapter 3 for the pressuremeter test, the borehole was first sunk to a depth about 4 feet above test elevation using a 7 inch diameter cone bit. The 3 inch diameter borehole was then advanced 7 feet using the specially prepared wing bit in the manner previously discussed. The probe was then lowered

into the borehole on "A"-rod usually achieving a good tight fit about 4.5 feet into the 3 inch diameter hole. Upon completion of the test the probe was withdrawn and the preparation cycle was repeated extending the borehole down to the next test elevation.

CH1 was prepared to a depth of 53 feet (bottom of the till sheet) by Mobile Augers Ltd., Edmonton. Ten inch diameter hollow stem, continuous flight augers were used for the primary drilling and a 3 inch diameter auger, lowered down the hollow stem of the larger auger, produced the test section of the borehole.

Test elevations were selected from the soil profiles obtained from the logs of boreholes CN2 and AGT2, with the object of obtaining at least one pressuremeter test in each distinct soil strata.

From the pressuremeter tests performed, at the three described locations, representative volume-time curves were prepared to determine the appropriate elastic time interval (Te) and consequently the elastic volume change ΔV associated with each pressure increment of each particular test.

The volume-time curves and their related volume-pressure diagrams, from which E is calculated, for boreholes CN1, CH1 and AGT1 are contained in Appendices A, B and C respectively. The relevant data and calculated E values from these Appendices are tabulated in Tables 4.1, 4.2 and 4.3.

Probably the most important result was the magnitude of the measured E values. They averaged nearly an order of magnitude greater than the UU triaxial results, measured on similar material sampled with the Pitcher sampler. In the case of the sand the best pressuremeter results were about 100% greater than the block sample moduli reported by DeJong (1971). This difference in modulus between laboratory and in

TABLE 4.1

PRESSUREMETER TEST RESULTS FROM BOREHOLE CN1

Soil Type	Depth (ft.)	Elastic Time Interval-Te (sec.) (see Fig. 3.5)	Vo-(cm^3)	$E=\frac{2Vo(1+\mu)\Delta P}{\Delta V}$ -(p.s.i.)	Comments
till	26.5	15	1180	16,900*	- All tests, $\mu = 0.40$
sand	33	15	1220	8,700*	- See Appendix A for data source
sand	39	15	1140	33,000	
till	52	120	1160	18,200*	* Vol.-time curves not sufficiently definitive for accurate calculation of E
till	60	30	1200	8,000*	
sand	67.5	30	990	35,500	
sand	78	15	1080	20,800**	**Air leak during test
sand	91.5	30	960	28,700	
shale	115.5	30	1150	49,500	
shale	122.5	15	920	33,000	
shale	148.5	15	1000	84,000	
shale	168.5	15	1060	128,000	

TABLE 4.2

PRESSUREMETER TEST RESULTS FROM BOREHOLE CH1

Soil Type	Depth (ft.)	Elastic Time Interval-Te (sec.) (see Fig. 3.5)	Vo-(cm^3)	$E=2Vo\frac{(1+\mu)\Delta P}{\Delta V}$ -(p.s.i.)	Comments
till(sandy)	20	30	1060	6,400	All $\mu = 0.40$
till	25	30	1030	21,100	*Conservative estimate of Te
till	30	30	1020	16,600	
till	35	30*	1025	27,100	
till	40	30*	1065	30,500	
till(sandy)	45	60	1110	6,200	
till	50	30	1040	20,000	

TABLE 4.3

PRESSUREMETER TEST RESULTS FROM BOREHOLE AGT1

Soil Type	Depth	Elastic Time Interval-Te (sec.) (see Fig. 3.5)	Vo-(cm^3)	$E=\frac{2Vo(1+\mu)\Delta P}{\Delta V}$ -(p.s.i)	Comments
sandy clay	52	60	1230	12,700	All μ= 0.40
fractured shale	71.5	30	1080	12,700	
fractured shale	84	60	1140	12,800	
fractured shale	94	30	1240	13,700	
shale	122	15	920	49,500	
siltstone	132	15	1030	104,600	
shale	144	15	1080	87,000	
shale	152	30	1160	115,400	
shale	169	15	1110	133,900	
shale	189	30	1120	130,500	

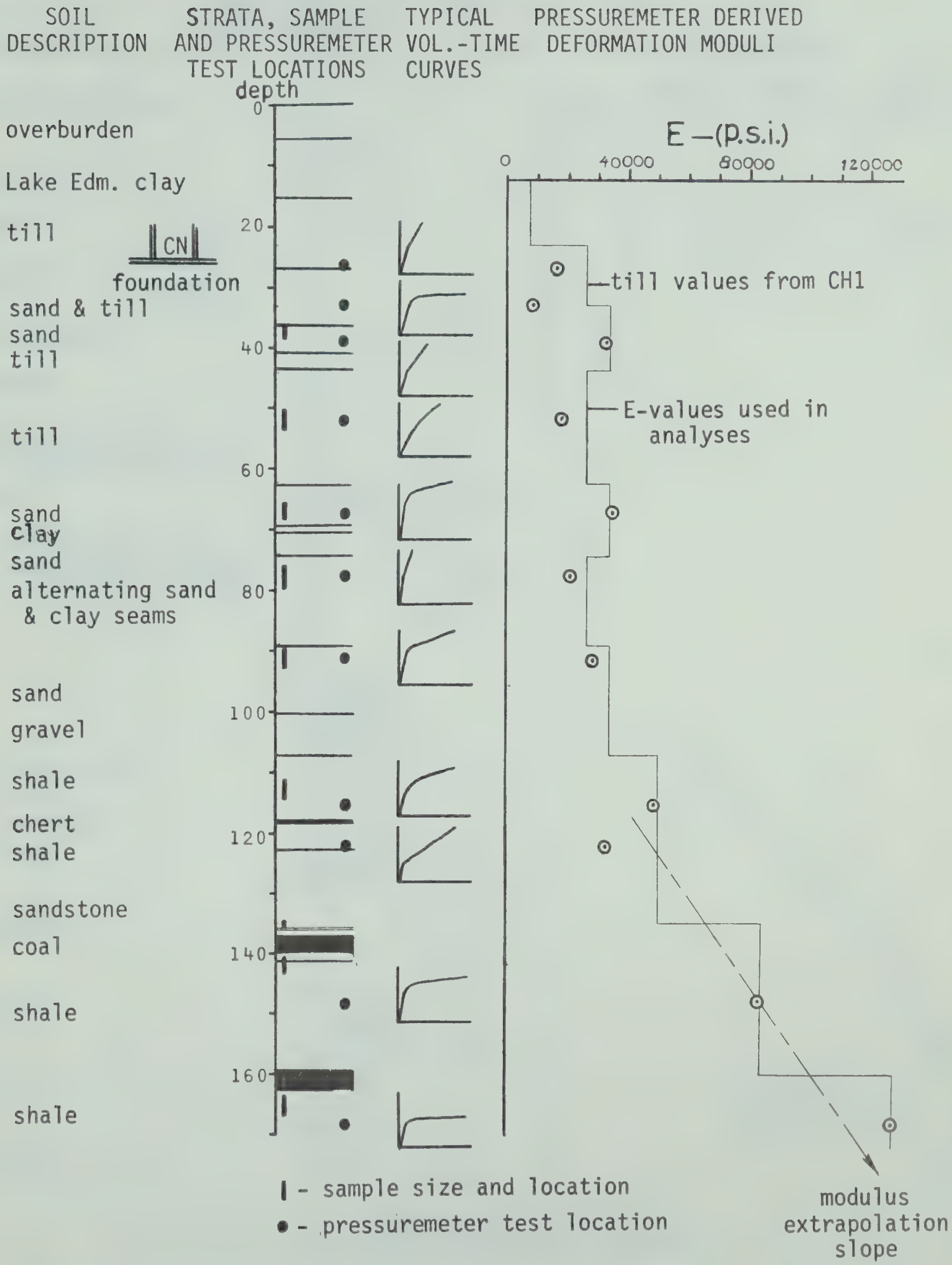

FIG. 4.3 FOUNDATION INVESTIGATION INFORMATION - CN TOWER

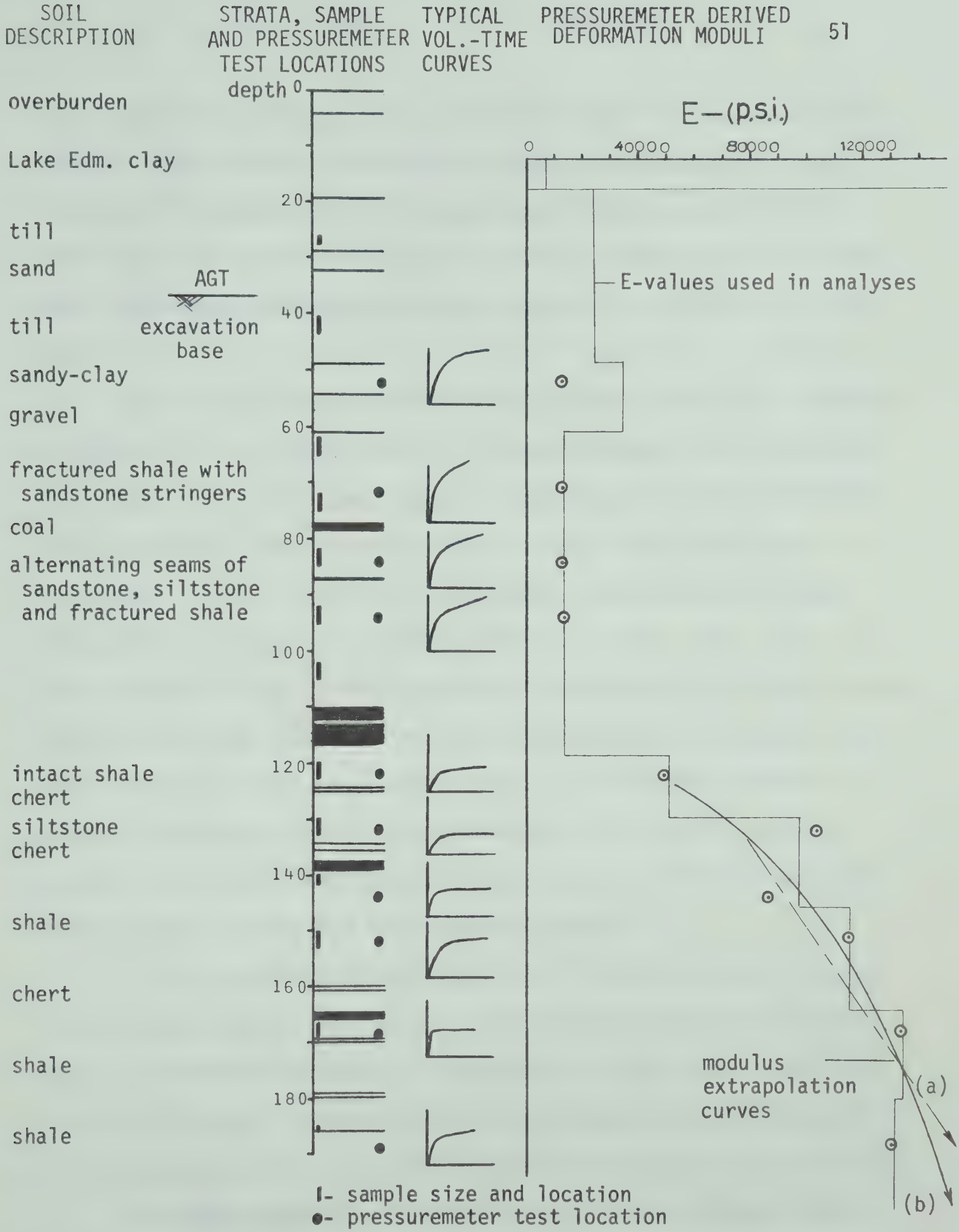

FIG. 4.4 FOUNDATION INVESTIGATION INFORMATION - AGT TOWER

situ measurement was encouraging on the basis that it represented approximately the calculated to observed settlement ratio of the two towers. Furthermore the magnitude of the pressuremeter derived moduli in the shale was of the same order as that reported by Underwood et al., (1964) from field rebound measurements on the similar Pierre Shales at the Oahe Dam site.

It is now generally believed that a uniform soil deposit exhibits an increased stiffness with depth. The pressuremeter results supported this hypothesis at both tower locations upon encountering intact bedrock (shale) deposits. The phenomenon began at about the 110 foot depth at both towers (Figs. 4.3 and 4.4) increasing at approximately the same average rate of 1400 p.s.i. per foot increase in depth. This result is most important in that it does not depend upon the ability of the pressuremeter to accurately determine the deformation modulus but instead on the reproducibility of the test. The accuracy of this modulus gradient is further supported by the increased confidence with which the elastic portion of the volume-time curves could be selected from the tests performed in both CN1 and AGT1 below 100 foot depths.

It is interesting to note from Fig. 4.4 that the moduli obtained from the pressuremeter tests in fractured shale is constant with depth and the corresponding volume-time curves do not have a well defined Te value. This behavior would suggest that the load-deformation behavior in the fractured shale is controlled essentially by its fissured structure.

The pressuremeter tests in sand at the CN tower produced moduli ranging from 8,700 - 35,500 p.s.i. Since there was no observable cementation in the sand encountered between the 30 - 100 foot depth at this location it was reasoned that the modulus value should be a function

mainly of the confining pressure. The stress history of the sand which included high compressive stressing should have produced a deposit of fairly uniform density that probably also contains residual stresses. It was therefore assumed that E should have been essentially constant or increased somewhat with depth and since it did not, one can assume that the test sections produced were of varying degrees of inadequacy with respect to probe requirements. Any amount of imperfection of the borehole should result in a reduction in measured moduli and therefore the largest value measured was the one considered most representative of the true sand modulus.

The volume-time curves (Appendix A) lend support to the decision to disregard the low sand modulus values.

The non-definitive nature of the volume-time curves for the till deposits at the CN tower prompted the substitution of those obtained in CH1. Of the till moduli only the high values were accepted for the same reasons as outlined in the selection of the sand moduli.

It was observed without exception that if the pressuremeter borehole was tight and the probe had to be forced into position, the volume-time curves of the test were well defined with respect to location of Te and the volume-pressure curves were linear. This tolerance requirement is a currently unfortunate necessity arising from the inadequate design of the measuring cell of the probe. Fig. 3.3 contains a possible remedial design.

An economical consequence of the pressuremeter tests is that the duration for which each volume-time curve is taken does not appreciably affect the shape or ΔV value of subsequent curves of the particular test. It is therefore unnecessary in principle to carry each volume-time curve

beyond Te. Since Te varies slightly with pressure level and pressure increment magnitude it would be wise to carry each volume time curve about 1 minute beyond the Te duration established earlier in the test.

It is extremely important, particularly in stiff soils, to take many volume-time readings in the first 60 seconds of each pressure increment as it is during this time interval that Te will be located.

Chapter III contains rules that should be observed in performing the pressuremeter test. An infraction of one of these rules is evident in the volume-time curves at the 52 foot test depth of CN1 (Appendix A). The curves demonstrate the result of not maintaining a measuring cell pressure at least 10 p.s.i. greater than the guard cell pressure.

About 1.3 hours was required to perform each complete pressuremeter test and this includes actual test and calculation time. The test time can be reduced according to the procedure outlined above but calculation time will remain at about 20 minutes per test.

The average drilling costs per pressuremeter test for the CN and AGT projects was $110. This cost includes the drilling of the borehole @ $24 per hour and the handling of the pressure probe @ $16 per hour. Material costs such as bits, drilling mud and damaged casing was extra. Also not included are the pressuremeter operator's wage.

This cost is very high in relation to the total cost per pressuremeter test of about $50 that was reported by Calhoon (1972). Although drilling effort increased with the depth of the borehole, the method with which the borehole had to be advanced generated most of the inflated cost. The proposed probe design and testing procedure discussed in Chapter III should yield at least a 50% reduction in cost per test.

Pitcher samples were taken from boreholes CN2 and AGT2 at

approximately the same depths at which the pressuremeter tests were performed (Figs. 4.3 and 4.4). The average drilling costs per Pitcher sample obtained was \$100. An additional cost of about \$30 per laboratory test would be necessary thereby bringing the cost of each modulus of deformation value to \$130 before consideration of the technician's wage during the drilling and sampling stage of the investigation.

Of the samples obtained from boreholes CN2 and AGT2 only those taken from the intact bedrock and about 30% of those taken from the till were suitable for triaxial testing. Test specimens could not be prepared from the sand, sandy till or fractured shale samples. Had sampling and laboratory testing been the prime method of these investigations additional boreholes would have been necessary in order to obtain an acceptable number of moduli to perform the deformation analyses.

The costs incurred in the pressuremeter investigation are quite acceptable when compared to that of an equivalent investigation in which Pitcher sampling followed by conventional laboratory testing is carried out. Further desirability of a pressuremeter investigation over conventional methods arises when considering the low degree of confidence associated with the deformation moduli obtained from conventional laboratory tests.

4.4 Laboratory Test Results

Borehole CN2 and AGT2 (Figs. 4.1 and 4.2) were drilled by Artesia Drilling Ltd. using a Failing 1500 rig and wet drilling procedure. The purposes for the CN2 and AGT2 were to accurately determine the foundation strata for pressuremeter testing and to obtain samples for comparative laboratory testing. It was deemed prudent to locate the boreholes close enough to the towers to be representative of the actual foundation strata

but not so close that that the tower loads may have altered the geotechnical properties of the soil. The final location was a matter of convenience and the depth of investigation was mainly an economic decision.

4.5 inch diameter samples, three feet long were retrieved with the University of Alberta modified Pitcher sampler.

The intention was to secure from each strata representative, intact relatively undistrubed samples, suitable for triaxial testing. Figs. 4.3 and 4.4 report the depth and soil type in which sampling success was encountered.

The main problems in sampling were those of recovery and sample disturbance arising from secondary design inadequacies.

Of 8 sampling attempts in till only 2 resulted in any recovery. In sand there was a 20% recovery and in shale and siltstone there was 60% recovery.

The rock contained in the till deterred sampling by crimping the cutting edge of the sample tube sometimes preventing further advancement but more often resulting in no recovery or recovery of only small damaged samples. Success in the Pitcher sampling of sand was not anticipated and that achieved was due to the presence of thin clay seams. Recovery in the bedrock was initially about 40% due to insufficient expansion upon stress relief. This was accommodated for by later use of sample tubes with smaller inside clearances. Subsequent zero recovery was attributed to cutting edge damage of the sample tube upon encountering bedrock of excessive induration.

Secondary design inadequacies augmented the low core recovery. Several cores recovered were of uneven cross-section suggesting rotation

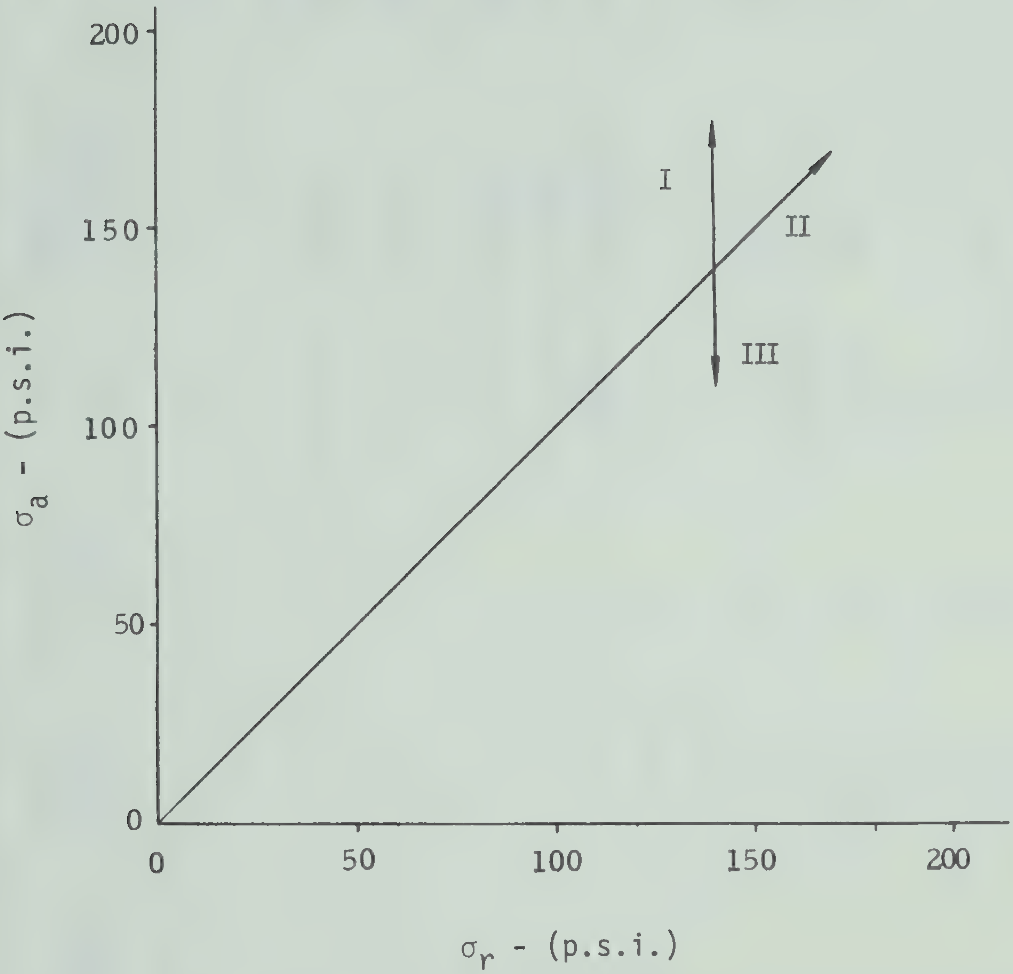

FIG. 4.5 STRESS PATHS USED IN TRIAXIAL TESTING FOR DEFORMATION MODULUS ON AGT1 SAMPLES

TABLE 4.4

AVAILABLE LABORATORY TEST RESULTS FROM
SAMPLES TAKEN AT THE AGT TOWER SITE

Soil Type	Depth (ft.)	Sample Type	Average Bulk Density (p.c.f.)	Water Content (%)	Degree Saturation (%)	Type of Test	Modulus of Deformation (p.s.i.)	Reference
lacustrine clay	5-20		119	28				DeJong (1971)
till			131					DeJong (1971)
sand	58	block				CAU Ko=.6	15,000	DeJong (1971)
sand	58	block				CAU Ko=.5	26,000	DeJong (1971)
sand	58	block				CAU Ko=.37	31,000	DeJong (1971)
sandstone	62	Pitcher				CAU Ko=.6	12,600	DeJong (1971)
shale	73			18.4		oedometer	2,330, μ=.4	DeJong (1971)
shale	73	Pitcher	134	17.8				
shale	83	Pitcher	133	18.4				
shale	90	Pitcher		17.5	85	UC	3,550	DeJong (1971)
shale	91	Pitcher		19.9		oedometer	3,410, μ=.4	DeJong (1971)
sandstone	92	Pitcher		16.7	94	UC	8,000	DeJong (1971)
shale	93	Pitcher	137	17.5				
shale	104	Pitcher		16.9				
shale	151	Pitcher	132	20.0	97	I (Fig.4.5)	17,000	
shale	151	Pitcher	132			II(Fig.4.5)	14,300 μ=.4	
shale	151	Pitcher	132			III(Fig.4.5)	8,800*	
shale	151	Pitcher	132	20.1	98	oedometer	24,600 μ=.4	

*load cell malfunction

of the sample tube and all samples had been fully exposed to drill fluid.

Of the samples obtained only the AGT2 samples taken from depths 73 to 151 feet inclusive have been subjected to partial laboratory testing.

The soil samples tested were prepared in the sample storage room which maintains a near saturated atmosphere at 40° F. The triaxial specimen were vertically oriented and the type of test indicated pertains to the corresponding stress-path indicated in Fig. 4.5.

The geotechnical properties obtained from the samples in question are given in Table 4.4 and have been supplemented by relevant data reported by DeJong (1971).

The 4-inch diameter triaxial samples were loaded isotropically under zero drainage to their estimated in situ overburden pressure. The samples were not consolidated to avoid a change in moisture content. The isotropic loading was held for about 4 hours at which time volume change had essentially ceased and the deviator stress was applied under undrained conditions.

The tests were stress controlled in which the axial stress was determined by a load cell, and the lateral stress was supplied by an air pressure source acting on a water reservoir which was in connection with the cell. Axial deformation was measured by a .0001 inch dial gauge and volume change of the sample under zero drainage was determined with a burette.

On the basis of the limited test data available, the following results appear evident.

a) Anisotropically consolidated undrained (CAU) tests performed on block samples of sand give E values slightly less

than pressuremeter moduli obtained in sand (for Ko = .37) but the variation in E with Ko from the CAU tests and the confidence with which one is able to select Ko makes it difficult to compare the two results decisively.

b) Deformation moduli measured on Pitcher samples vary appreciably depending on the type of test and the stress path taken to failure but all have the common characteristic of being 1/5 to 1/7 the magnitude of the deformation moduli obtained from the pressure-meter tests.

c) From the degree of saturation measurements the water table would appear to be near the 150 foot depth elevation.

4.5 Summary

The AGT and CN towers are located on the north side of the North Saskatchewan River in downtown Edmonton. The AGT tower is located about 200 feet away from the river bank whereas the CN tower is situated about 1/2 mile north of the AGT tower at approximately the same ground elevation.

Similar soil profiles exist at both towers with local differences with respect to depth of individual deposit.

The stratigraphic sequence in descending order consists of: silty lacustrine clay; sandy till containing sand lenses; medium to fine grained sand underlain by several feet of gravel; and interbedded bentonitic mustones (shale) and silty sandstones with occasional seams

of chert and lignite coal. Borehole logs taken in the river valley about 180 feet below and 1/2 mile south of the AGT tower indicate similar bedrock composition (Matheson, 1972).

The foundation investigation of the two towers consisted of in situ pressuremeter testing for deformation modulus and to a lesser extent laboratory testing on soil specimen obtained with the Pitcher sampler.

Two boreholes were drilled near each tower location, one used for sampling and logging strata and the other used for pressuremeter testing.

Significant information both in regard to moduli measured and pressuremeter test behavior, originated from the pressuremeter investigations and are listed below.

a) The magnitude of the pressuremeter derived moduli was much greater than the comparable laboratory values reported by DeJong (1971) in his investigation of the two tower foundations, the difference approaching an order of magnitude.

b) Upon encountering intact bedrock the pressuremeter obtained moduli showed a definite dependence on confining pressure. The deformation modulus increased with depth at approximately the same rate for the two sites investigated. Insufficient tests were performed to accurately establish the nature of the modulus gradient and therefore a linear variation was assumed in the analyses of the CN Tower foundation (Fig. 4.3). The moduli obtained at the AGT Tower was better suited for a non-linear extrapolation of E with depth (Fig. 4.4, modulus extrapolation slope (b)) than those obtained at the CN Tower and therefore

both linear and non-linear increase in E with depth was considered in the heave analyses.

c) Pressuremeter tests in the fractured shale at the AGT Tower site indicated the anticipated macrostructural control of the deformation modulus. The volume-time curves showed elastic-plastic like behavior at low stress levels which can readily be explained in terms of deformation response to changes in applied stress resulting from the closing of fissures.

d) In pressuremeter testing of easily disturbed soils, if the deposit is known to be relatively homogeneous, then it may be acceptable policy to regard only the highest measured E values as being representative of the in situ stiffness of the soil. Attention must be given to the possibility of increasing modulus with depth and the acceptability of the volume-time curves from which the respective moduli were calculated before making the decision of acceptance or rejection of modulus values.

e) There appears to be no need in performing volume-time measurements beyond Te for each pressure increment but many readings should be taken during the first 60 seconds of each pressure increment.

f) Due to probe design inadequacies that required time consuming drilling procedures, the cost per pressuremeter test was excessive, however when compared to a conventional investigation involving sampling and laboratory testing, the cost per pressuremeter test becomes attractive.

g) Limited laboratory results indicate that deformation modulus is sensitive to the stress path taken to failure. However it appears that regardless of the stress path taken, the initial tangent modulus will be low by a factor of from 5 to 7.

CHAPTER V

FINITE ELEMENT ANALYSIS OF DEFORMATION AT THE CANADIAN NATIONAL (CN) AND THE ALBERTA GOVERNMENT TELEPHONES (AGT) TOWERS

5.1 Introduction

The soil stratigraphy underlying the AGT and CN towers is diversified due to depositional history and anisotropic due to preconsolidation.

The bedding of the soil strata in the upper 140 feet at the AGT site is shown in Fig. 5.2 for the section and corresponding boreholes indicated in Fig. 5.1. The bedding is observed to be fairly level under the excavation site and has been assumed to remain so for a radial distance of up to 900 feet.

Soil stratigraphy below the CN tower as reported by DeJong (1971), for the five testholes indicated in Fig. 4.1 , is also sufficiently uniform in elevation to give little error if assumed level in the foundation analysis. Testhole, CN2, (Fig. 4.1) lends further support to the assumed zero degree inclination of the strata and again this condition is assumed to exist for a 900 ft. radial distance from the center of the foundation area.

The soil below the depth investigated by boreholes CN2 and AGT2 was assumed to be shale and this seems to agree favorably with borehole results reported by Matheson (1972) at the James MacDonald Bridge site on the North Saskatchewan River about 1/2 mile from the AGT site.

The complex nature of the soil below both towers necessitated the use of the finite element technique for the foundation deformation

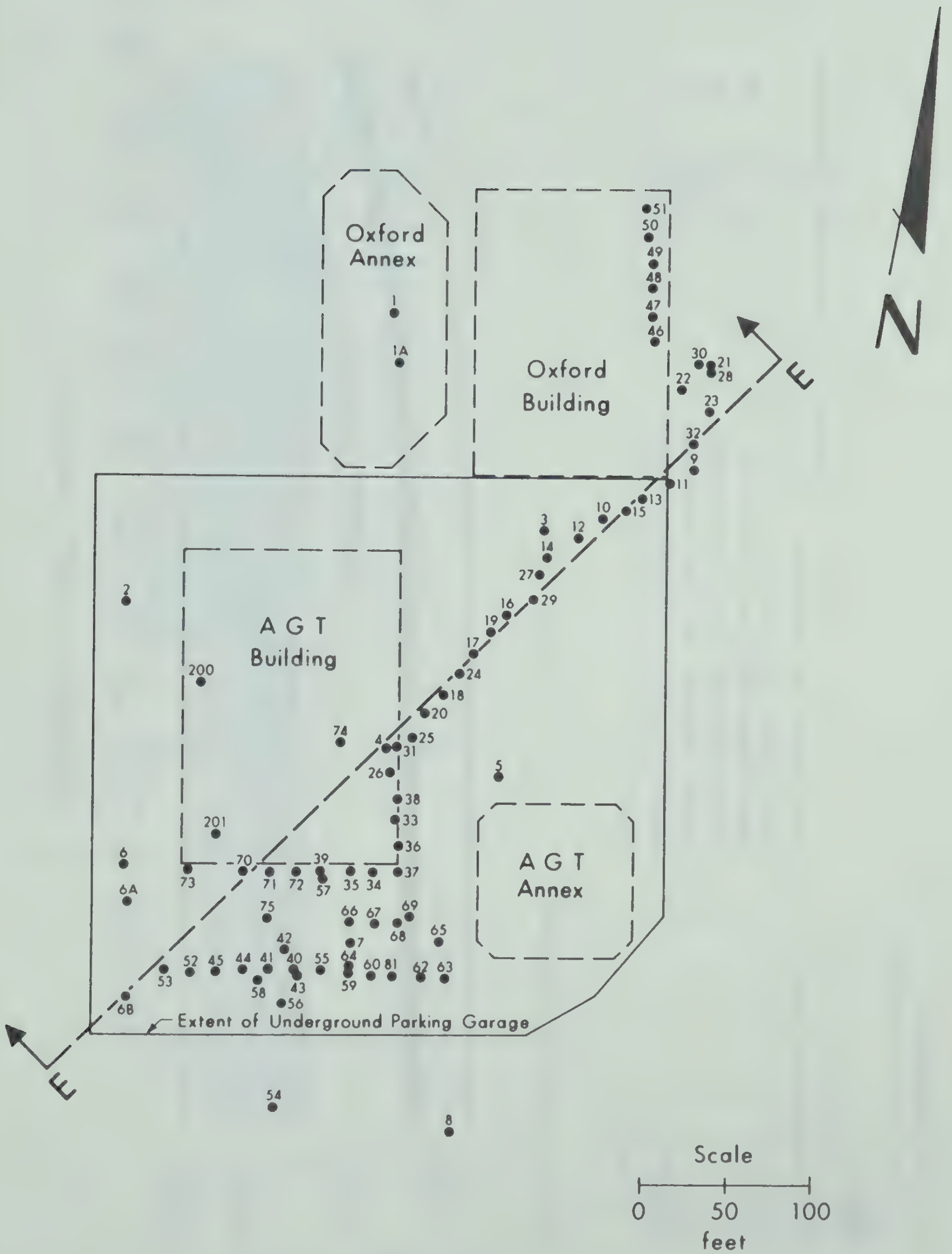

FIG. 5.1 LOCATION OF BOREHOLES AND STRATIGRAPHIC SECTION - AGT TOWER (FROM DEJONG, 1971)

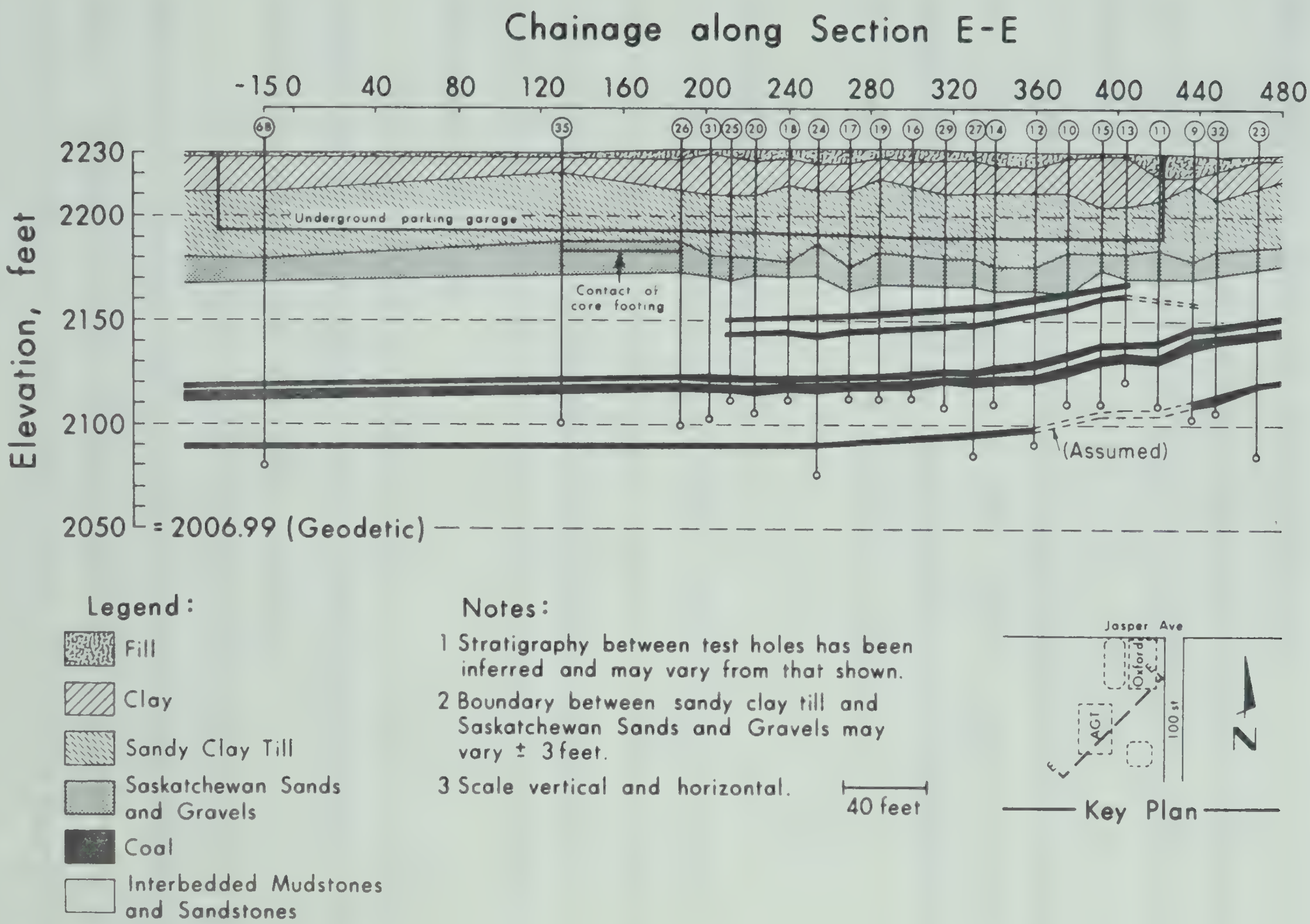

FIG. 5.2 STRATIGRAPHIC SECTION - AGT TOWER (FROM DEJONG, 1971)

analyses.

Settlement analysis was performed on the CN foundation and heave analysis was carried out for the AGT excavation.

Foundation loading and settlement data for the CN tower and the excavation unloading and heave data for the AGT site are those given by DeJong (1971).

Deformation moduli used were those determined from pressuremeter tests performed in boreholes CN1, AGT1, and CH1. Moduli for depths greater than boreholes CN1 and AGT1 were based on extrapolation of the modulus values obtained in the respective boreholes.

5.2 Finite Element Approach

Two and three-dimensional finite element analyses based on linear elasticity were used for the foundation settlement predictions of the CN Tower. The three-dimensional analysis was performed to ensure that the plane strain assumptions of the two-dimensional analyses were sufficiently satisfied.

Based on information obtained from the settlement analyses of the CN Tower, only two-dimensional (plane strain) analyses were used to predict base heave of the AGT Tower excavation.

The two-dimensional finite element program used was a modified version of the constant strain triangular element developed by Wilson (1963). The modifications included element and node generation routines and direct solution of displacements through the use of Gaussian elimination.

The three-dimensional finite element program used for verification was developed at the University of Alberta by A.V.G. Krishnayya and

employed 8-node isoparametric hexahedral elements. The remarkable feature of this analysis is its facility to accommodate all the normal and shear stress components as they influence deformation. Its limiting feature is the great demand on computer storage and data preparation.

In both the two and three-dimensional finite element analyses, each foundation stratum was assumed to be isotropic with respect to deformation properties. A value of E and μ was assigned to each grid layer of each stratum and as the result of the assumption of linear elasticity, deformations were calculated on the basis of change in stress without regard to stress level. A value of 0.4 was chosen for μ because the upper region of the foundations was not fully saturated and the immediate deformations associated with a given load increment could not be determined precistly.

5.3 Analysis of Foundation Settlement at the CN Tower

Fig. 5.3 shows a plan view of the relative location of the spread footings with respect to the central mat footing of the CN foundation. These footings are founded at an average depth of 23 feet and the perimeter of the footing group describes a rectangle 158 feet long by 110 feet wide.

These dimensions were at first considered to be too small with respect to plane strain requirements and therefore a three-dimensional analysis was also performed to establish the relevance of two-dimensional analysis for loadings of this areal extent.

The three-dimensional finite element gtid employed regular hexahedral elements, their number and relative dimensions and location controlled by the thickness of each soil strata, the location and extent of

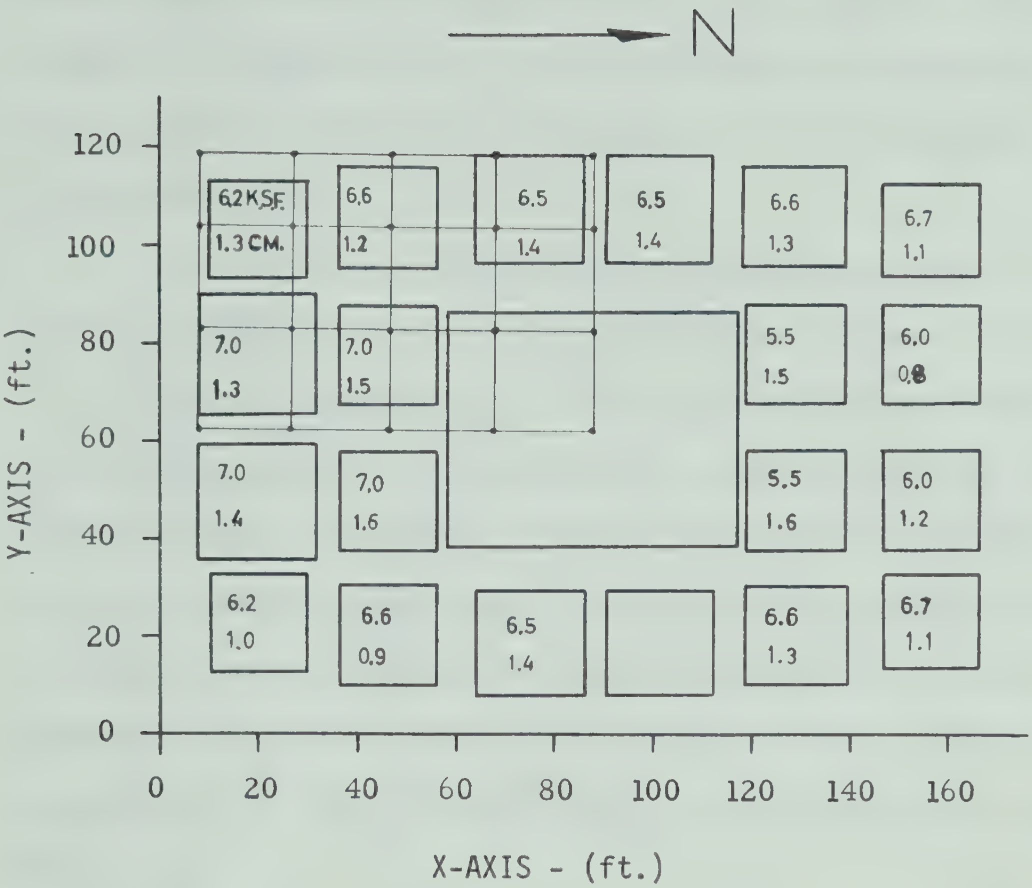

FIG. 5.3 FOOTING PLAN OF CN TOWER SHOWING NET APPLIED DEAD LOADS WITH THEIR CORRESPONDING SETTLEMENTS PLUS THE SUPERIMPOSED LOADING SECTION OF THE THREE - DIMENSIONAL FINITE ELEMENT GRID IN THE SOUTH-WEST QUADRANT

loading area, and the storage capacity of the computer. The grid thus chosen is displayed in Fig. 5.4 and represents in perspective the south-west quadrant of the CN tower foundation. Its dimensions are 320 x 400 by 363 feet deep.

Fig. 5.5 shows a magnification of the loading area of the grid with the respective footings superimposed for the sake of delineation.

The two-dimensional grid used in the plane strain comparative analysis was located at, and bounded by, the north face of the three-dimensional grid. The mesh, composed of triangular elements, had vertical dimensions that were controlled by the thickness of the foundation strata as in the three-dimensional grid but used 20 foot horizontal spacing to reduce errors arising from the use of disproportioned elements and to facilitate the node and element generation subroutines (Fig. 5.6).

Another two-dimensional grid 720 feet wide by 1003 feet deep was used to ensure a more complete account of deformation. It served for comparison between the best available three-dimensional analysis for this type of problem (Clough, 1969).

As important as the method of analysis is the accuracy of the input data, if there is to be confidence in the results. A critical examination of CN tower load deformation data is therefore necessary.

At the CN tower the dead loads as usual were more precisely known than the live loads. There was a lack of initial settlement measurements as the settlement plugs could not be established until the footings had been constructed. It also was necessary to terminate settlement readings at 5 columns at the end of dead load application. Further no columns were instrumented for the purpose of measuring the load they

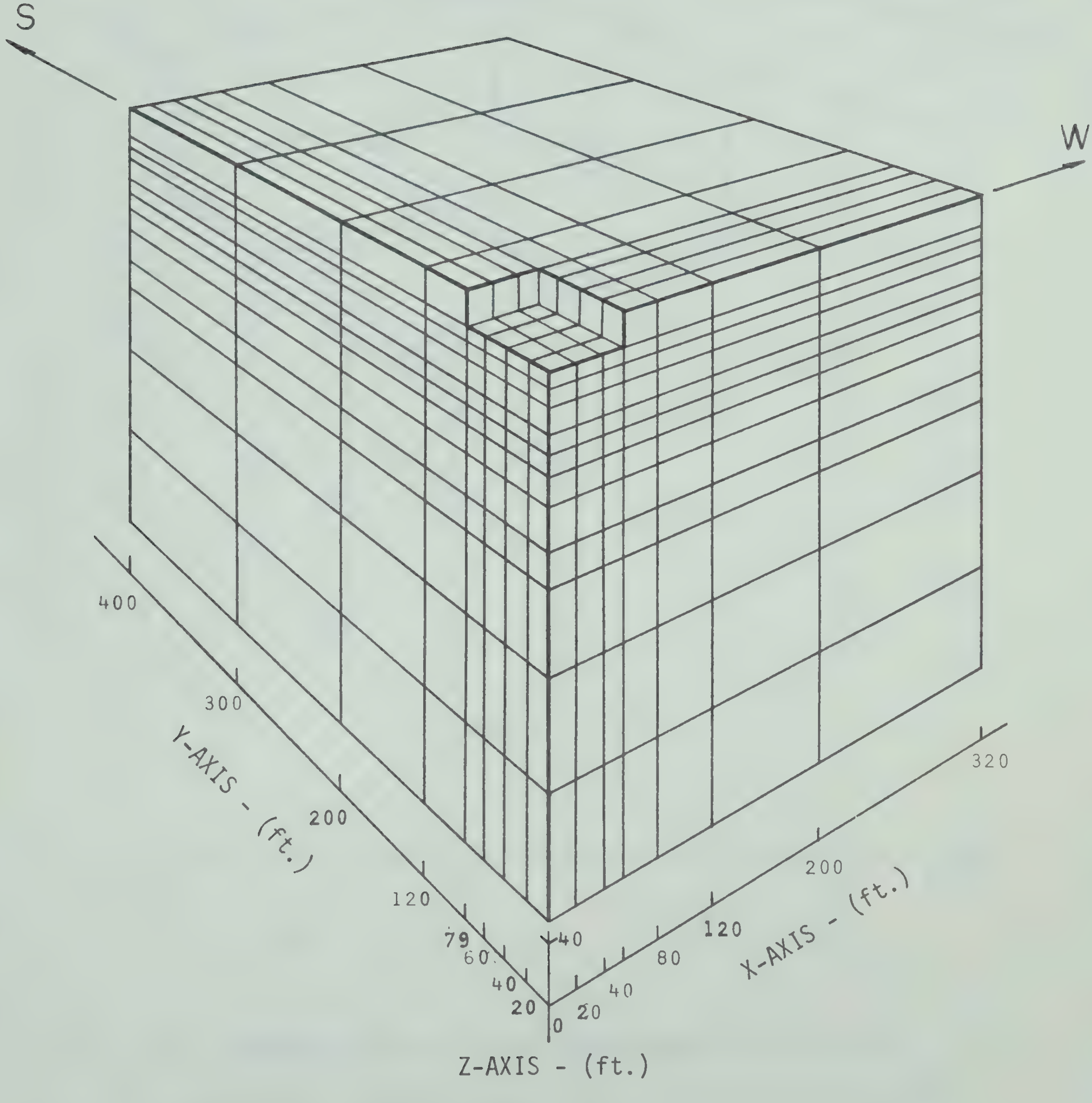

FIG. 5.4 THE THREE-DIMENSIONAL GRID USED IN THE FINITE ELEMENT ANALYSIS OF THE CN TOWER

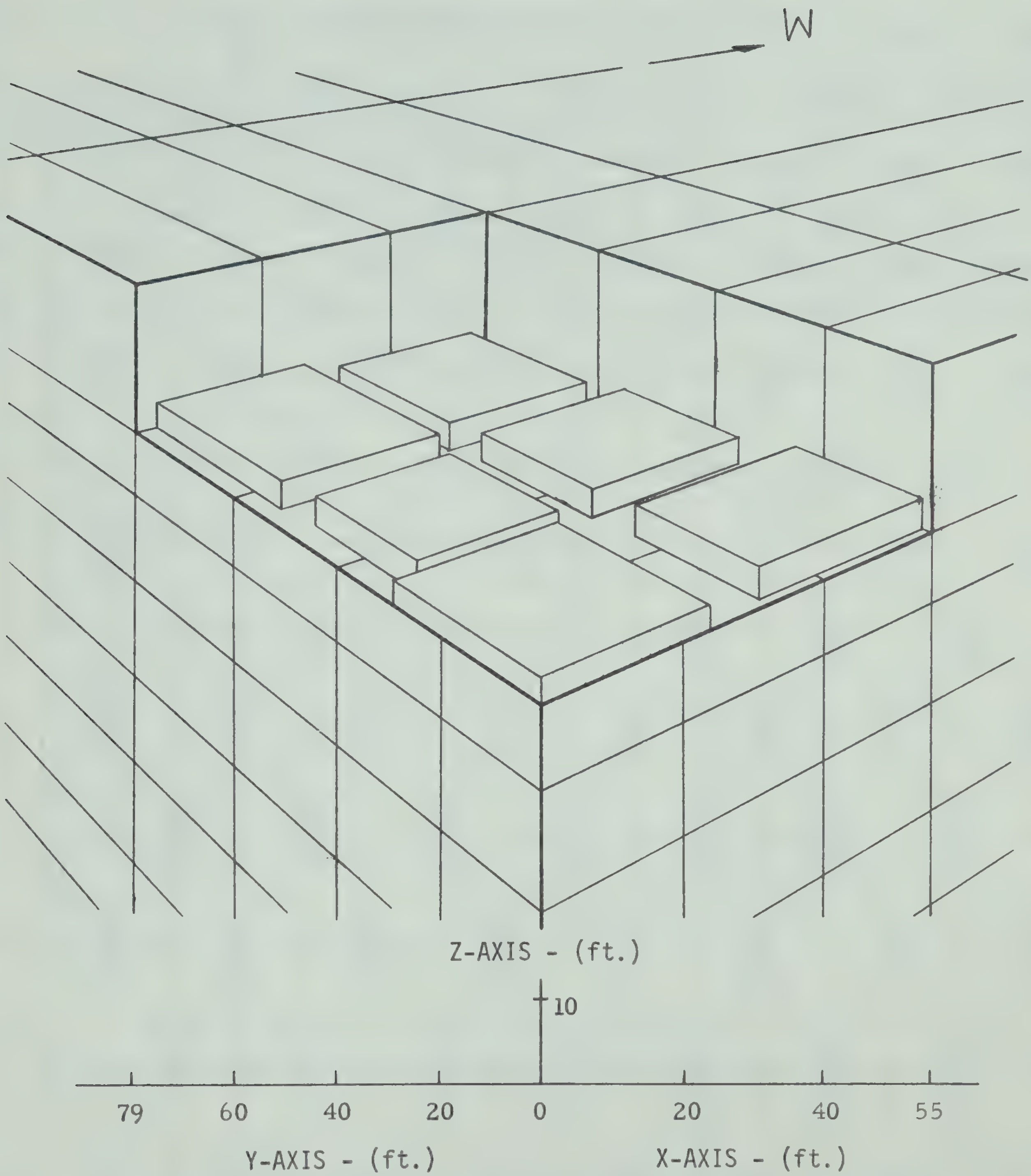

FIG. 5.5 LOADING SECTION OF THREE-DIMENSIONAL FINITE ELEMENT GRID WITH SUPERIMPOSED FOOTINGS - SOUTH-WEST QUADRANT OF CN TOWER FOUNDATION

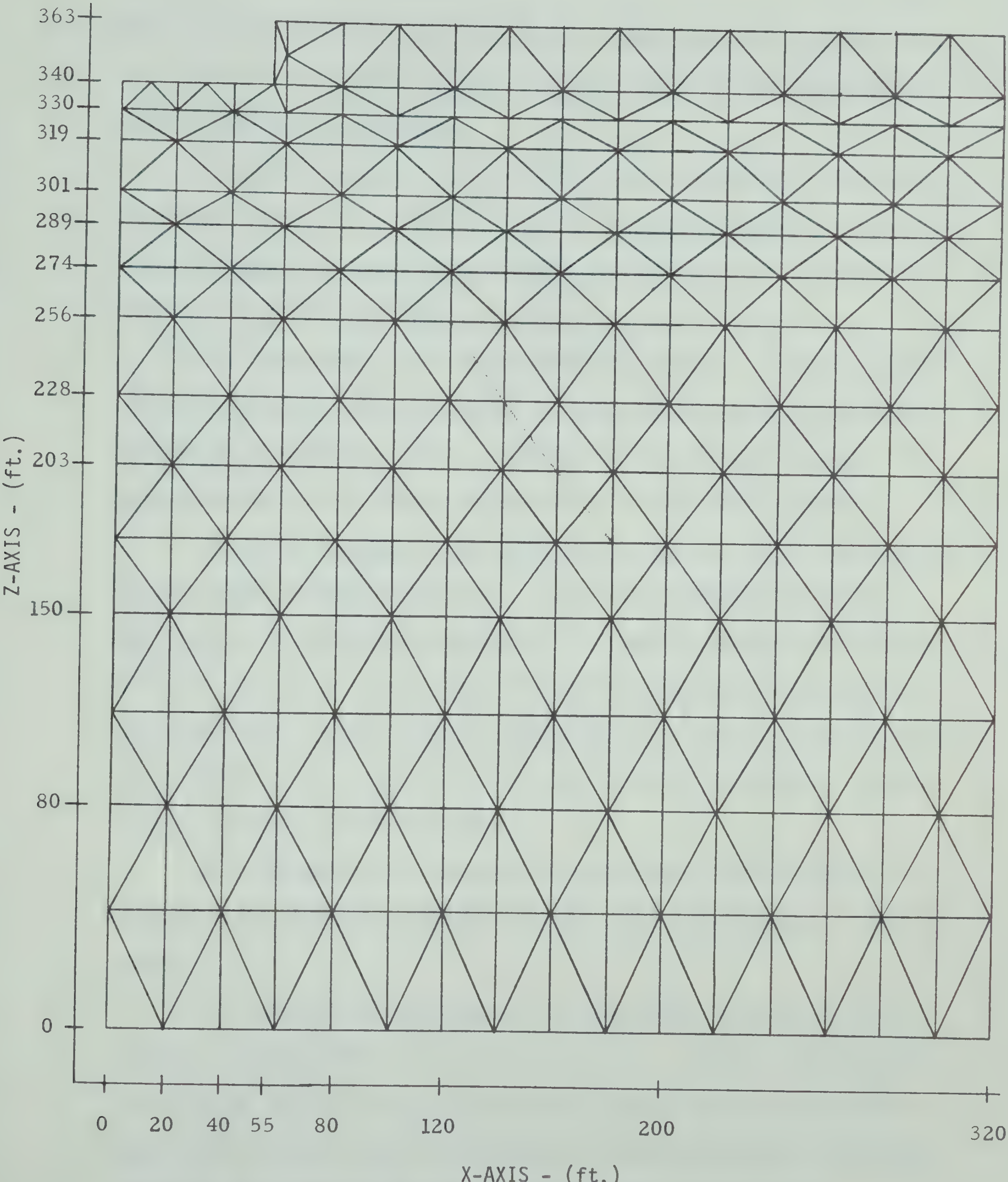

FIG. 5.6 SMALLEST TWO-DIMENSIONAL FINITE ELEMENT GRID USED IN CN TOWER FOUNDATION ANALYSIS

carried (DeJong, 1971) and therefore the loads reported are those based on calculated dead weight that have not been modified for load transfer under differential settlement.

With these facts in mind, as well as the symmetrical loading requirement of the finite element analysis, as much ambiguity as possible was eliminated from the load settlement data and analysis in terms of average net applied deadload was adopted.

The assumption in the finite element analysis, is that the load distribution across the foundation, remains constant under differential foundation deformation. This is similar to the flexibility approach used by DeJong in his load calculations at the different footings.

It should be kept in mind that this assumption is an idealization of the actual foundation behavior. The actual foundation undergoes many cycles of differential settlement followed by moment redistribtuion of loads which give rise to new differential settlement. The redistribution of loads to each column is controlled by a complex interrelationship of stiffness effects that is further complicated by hysteresis behavior in the structure and the foundation.

This assumption of flexible load application should tend to be conservative, predicting somewhat greater total and differential settlements.

The net applied deadloads and corresponding settlements as calculated by DeJong (1971) are shown in Fig. 5.3. Loadings were assumed for footings where load was not reported. These loads were then averaged over the whole area giving rise to a uniform stress of 4.435 k.s.f. which was then used in the two and three-dimensional analyses.

All the values of deformation moduli used in the CN foundation

analyses, came from pressuremeter tests in borehole CN1 except those obtained in till. Tests performed on till at the south-west corner of the old courthouse, (borehole CH1) located about 1000 feet south of the CN tower, were more definitive and could be used with greater confidence than those obtained in CN1. CH1 was dry augered whereas CN1 was wet drilled perhaps creating a more irregular borehole.

Each layer of elements was assigned a modulus value for the analyses according to the soil type it represented. Element layers situated below 170 feet were assigned modulus values based on the pressuremeter moduli extrapolated linearly to the center of each respective layer (Fig. 4.3).

A Poisson's ratio of 0.4 was assigned to all elements except for two analysis where the effect of the value of Poisson's ratio on deformation was being determined.

Table 5.1 below gives pertinent information as to the analyses performed and the results plotted for the CN tower foundation.

TABLE 5.1

ANALYSES PERFORMED AND RESULTS PLOTTED

FOR THE CN TOWER FOUNDATION

Type of Finite Element Analysis	Grid Dimensions-ft. Depth	Width	Length	Poisson's Ratio	Results Plotted	Figures Describing Data
3-D	363	320	400	0.4	Settlement at foundation level along the North face	5.7
3-D	363	320	400	0.4	Settlement of the loaded area	5.8
2-D	363	320	-	0.4	Settlement at foundation level	5.7, 5.9
2-D	1003	720	-	0.4	Settlement at foundation level	5.7
2-D	363	320	-	0.3	Settlement at foundation level	5.9
2-D	363	320	-	0.495	Settlement at foundation level	5.9

From Fig. 5.7 it is evident that, for a uniform loading over an area of 158 x 110 feet, on a nonhomogeneous stratified foundation, the two-dimensional finite element analysis is as accurate as the three-dimensional analysis for predicting settlement in the plane strain regions

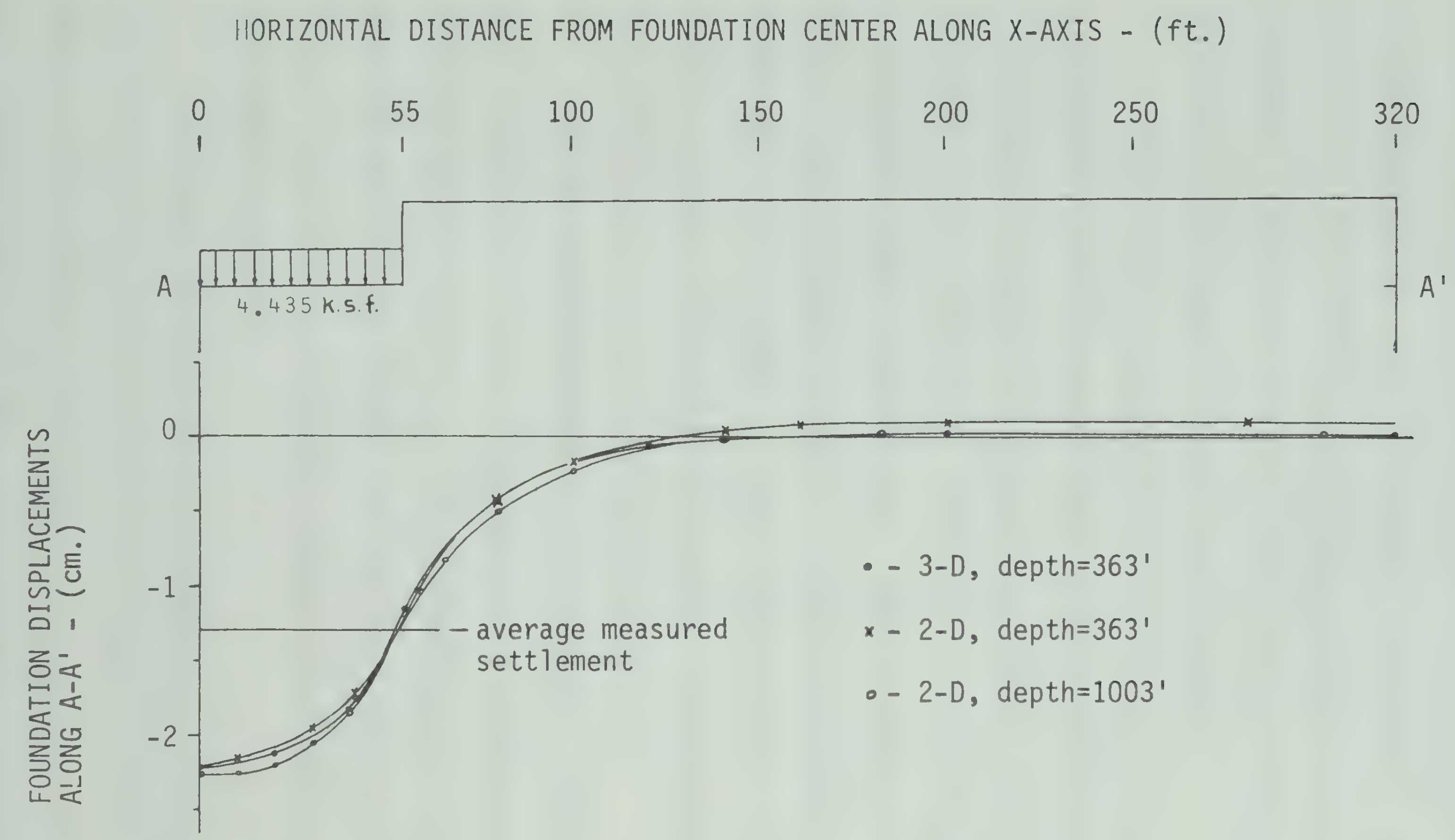

FIG. 5.7 FOUNDATION DISPLACEMENT PROFILES AT THE CN TOWER AS DETERMINED BY TWO AND THREE-DIMENSIONAL FINITE ELEMENT ANALYSES

of loading. There is a very small deviation of the two-dimensional from the three-dimensional analysis in the region of high shear stress as was expected but this difference is negligible. The disparity is due mainly to different strain representation within the respective elements. This error could be further reduced through the use of more elements in the region of high stress gradient.

Since it was necessary to work in terms of average stresses and settlements, the three-dimensional analysis has singular applicability with respect to average settlement comparison. Fig. 5.8 shows the loading section of the three-dimensional finite element grid with the corresponding nodal deformations. These deformation (settlement) values when averaged were found to exceed the average measured settlements (Fig. 5.3) by only 7%. Nodal deformations corresponding to the central mat were not averaged in because settlement measurements had not been taken on this foundation component.

The maximum differential settlement over a 30 foot span was 0.6 cm. for both the two and three-dimensional analyses whereas the measured maximum was 0.7 cm. In considering this differential settlement one should keep in mind that the real loading was not uniform nor fully flexible and so should be regarded more critically than the average settlement comparison.

There was approximately a 5% increase in the settlement profile in going from a grid depth of 363 feet to 1003 feet thus indicating little need for considering a depth greater than about 4 times the width of the loaded area for this particular foundation analysis. This relative insensitivity of settlement, with respect to depth of rigid base, for a given foundation loading is due primarily to increase in deformation

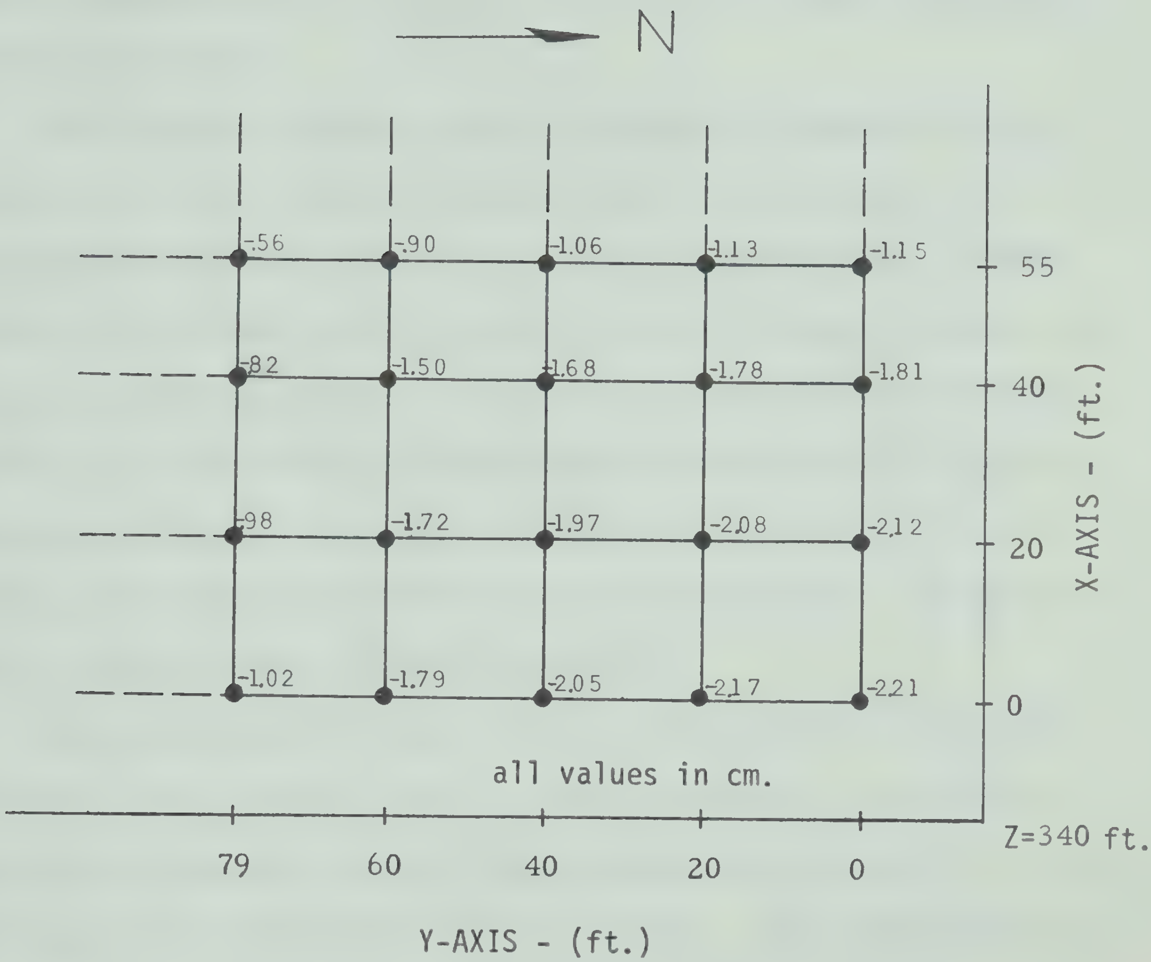

FIG. 5.8 CALCULATED NODAL SETTLEMENTS - LOADING SECTION OF THREE-DIMENSIONAL GRID - SOUTH-WEST QUADRANT, CN TOWER FOUNDATION

modulus with depth. If the project will allow the increased cost, several depths should be considered since the actual depth at which the settlement (or heave) will become asymptotic will depend upon the loading intensity and aerial extent and the rate at which the modulus increased with depth.

The value of Poisson's ratio is relatively unimportant in the calculation of E from the pressuremeter test since it enters into the formulation in the form of $(1 + \mu)$ as shown in Equation 3.1. However in the case of the finite element analysis, the settlement results are sensitive to Poisson's ratio. Fig. 5.9 shows graphically the effects of Poisson's ratio on the settlement magnitude and profile in the two-dimensional analysis. The effect of increasing Poisson's ratio is to decrease the maximum settlement in a non-linear fashion. The settlement profile however remains relatively unaltered.

Upon increasing Poisson's ratio, a greater lateral strain must accompany a given vertical strain. Lateral movement is prevented at the vertical boundaries of the finite element grid to simulate the lateral confinement that exists in the field situation. This restricted lateral movement results in higher lateral compressive stresses being developed for a given vertical stress increment and therefore according to Hooke's Law the vertical strain is reduced.

The non-linear effect of Poisson's ratio on settlement arises from the fact that under plane strain conditions, vertical strain is a non-linear (cubic) function of Poisson's ratio.

Selecting $\mu = .42$ results in convergence of computed and measured average settlement but a divergence of differential settlement. This divergence of differential settlement, though not conclusive evidence,

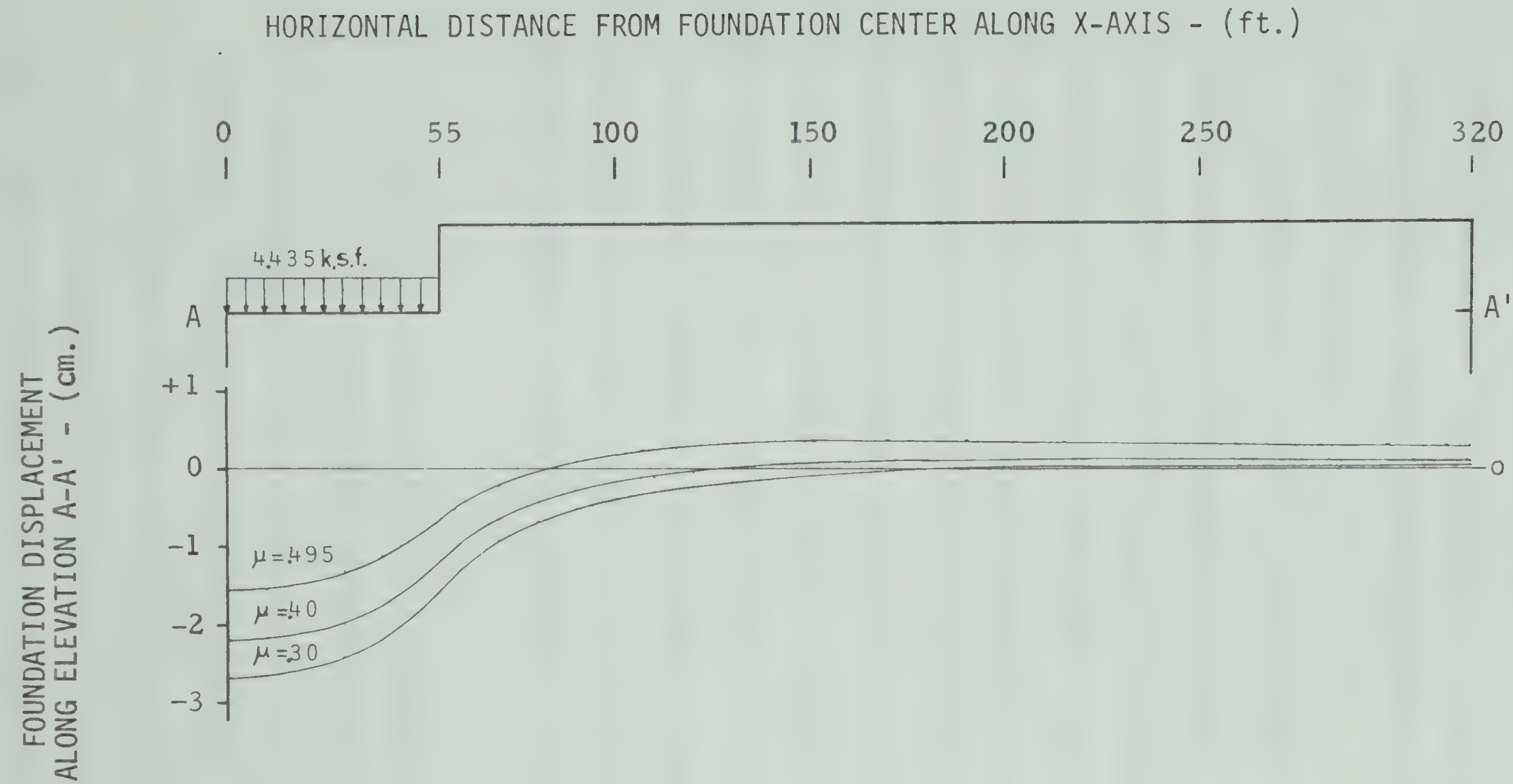

FIG. 5.9 EFFECT OF POISSON'S RATIO ON FOUNDATION DISPLACEMENT FOR THE TWO-DIMENSIONAL FINITE ELEMENT ANALYSIS - CN TOWER

along with the nature in which the load-settlement data were obtained, supports the use of Poisson's ratio less than 0.5. About 220 days are involved in the settlement readings during which the load was applied continuously and therefore totally undrained load-settlement data were impossible to obtain.

Assuming $\mu = .42$ to be correct then it can be concluded that the pressuremeter moduli are exceptionally good. However the possibility of modulus anisotropy due to overconsolidation should not be forgotten, in view of the fact that the pressuremeter measures modulus normal to the borehole axis and this lateral modulus of deformation may be somewhat higher than the vertical modulus of deformation. Soil disturbance at the borehole fact, softening due to wet drilling procedure and the degree of lateral stress relief due to the downcutting of the North Saskatchewan River should serve to decrease somewhat the anisotropic effect of overconsolidation. However a more rigorous account of possible modulus anisotropy at the CN and AGT Tower sites is desirable. Although acknowledged, possible anisotropy is disregarded in the calculations.

5.4 Analysis of Heave at the AGT Excavation

Fig. 5.10 shows a plan view of the AGT excavation with location of rebound points and excavation zones. Also shown is the sequence of excavation. The excavation was approximately 330 feet square by 37 feet deep and was situated about 50 feet away from the river bank.

Two-dimensional analyses with a Poisson's ratio equal to 0.40 were chosen on the basis of experience gained in the CN analysis. Close proximity to the river bank suggested a possible influence of the bank on the results of the analyses. Therefore plane strain analyses were

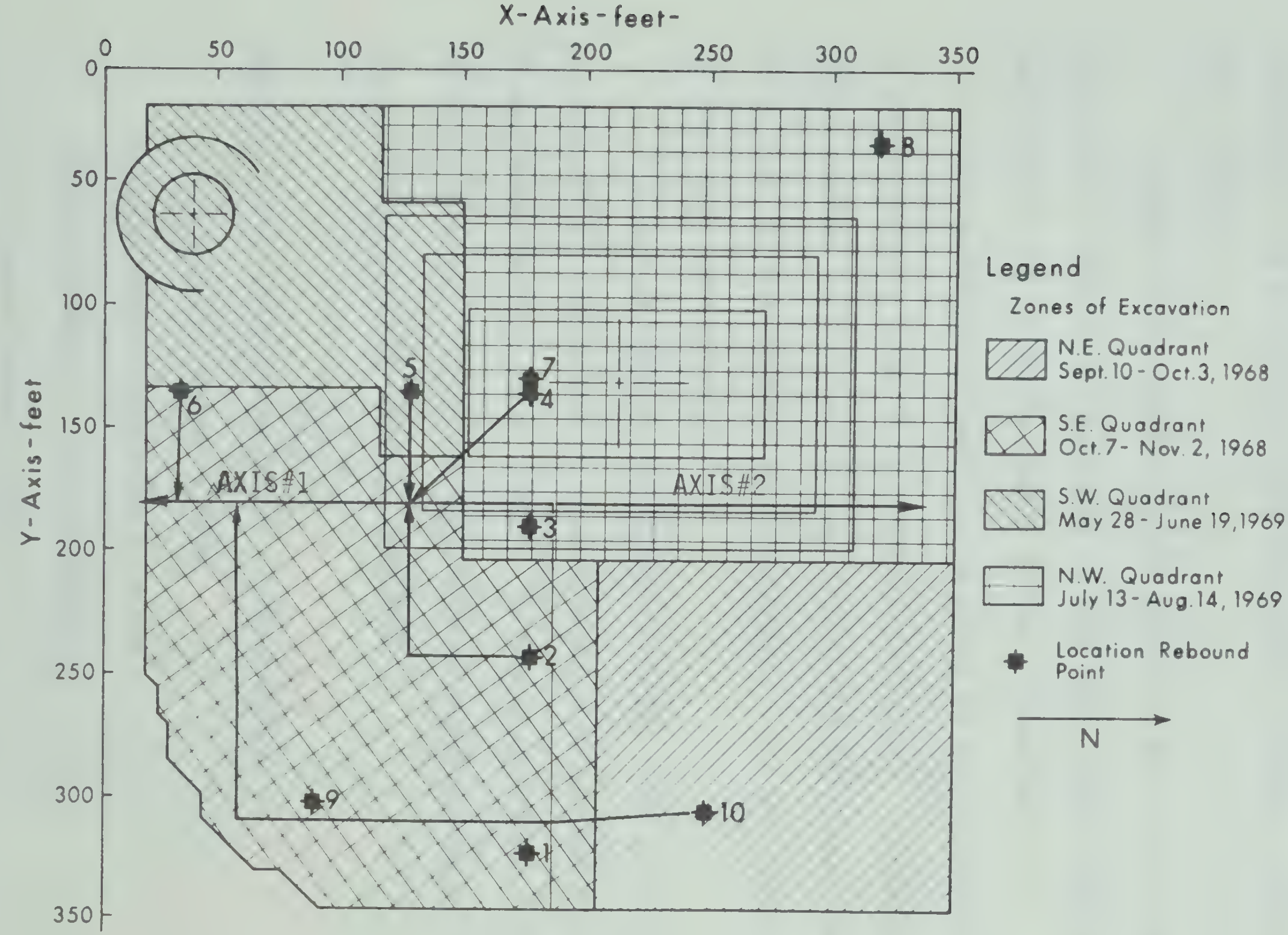

FIG. 5.10 LOCATION OF EXCAVATION ZONES AND REBOUND POINTS - AGT EXCAVATION (AFTER DEJONG, 1971)

performed along axes number 1 and 2 as indicated in Figs. 4.2 and 5.10. The surface profile along axis #1 (Fig. 5.14)) was determined from a contour map obtained from the Engineering Department of the City of Edmonton. The surface profile along axis #2 was assumed horizontal (Fig. 5.13).

Plane strain conditions are better satisfied along a north-south axis than an east-west axis due to the river bank running essentially parallel to the southern border of the excavation.

All deformation moduli were based on pressuremeter values obtained in borehole AGT1 (Fig. 4.2). Moduli below that investigated with the pressuremeter were determined by linear and non-linear extrapolation (Fig. 4.4) of the modulus values obtained from the pressuremeter tests in the lower part of AGT1.

The non-linear extrapolation used was similar to the power law relationship between the initial tangent modulus and the minor principal stress described by Janbu (1963). A "reasonable" non-linear projection of curve (b) Fig. 4.4 below the 190 foot depth produced a curve that could be described by Janbu's equation

$$E = C\,\sigma_a \left(\frac{\sigma_3'}{\sigma_a}\right)^n \qquad 5.1$$

where E = the undrained modulus of deformation

C = a modulus number = 2300

σ_a = atmospheric pressure = 14 psi

σ_3' = the effective lateral stress

n = exponent number = 0.67

σ_3' was estimated on the basis of a single value of Ko = 1, a density of 132 lb/ft.^{3}, and the water table was assumed to be located at a depth of 130 feet.

Excavation was carried out in the manner indicated in Fig. 5.10 with heave measurements taken as shown in Figs. 5.11 and 5.12. Stress release was based on the total unit weight of the soil removed as determined by DeJong (1971) and was equal to 5.0 K.S.F. along the base of the excavation. The measured heave values at each rebound point were based on superposition of the elastic response experienced at each respective rebound point during each partial excavation. The rebound points were projected on to the plane strain axis as indicated in Fig. 5.10. The measured heave values in which the rebound at points 2, 4 and 5 have been averaged, are plotted in Fig. 5.13. Points 6 and 10 are projected slightly toward the excavation boundary to account for the smaller displacement gradients along planes parallel to the axes of symmetry. Points 2, 3, 4 and 5 lie within the region of low displacement gradients and therefore were projected orthogonally.

DeJong reports difficulty in monitoring rebound point number 3 after excavation of the S.E. quadrant, possibly due to lateral movement. This may have contributed to its relatively low heave value.

Five analyses were performed on the AGT excavation and are described in Table 5.2.

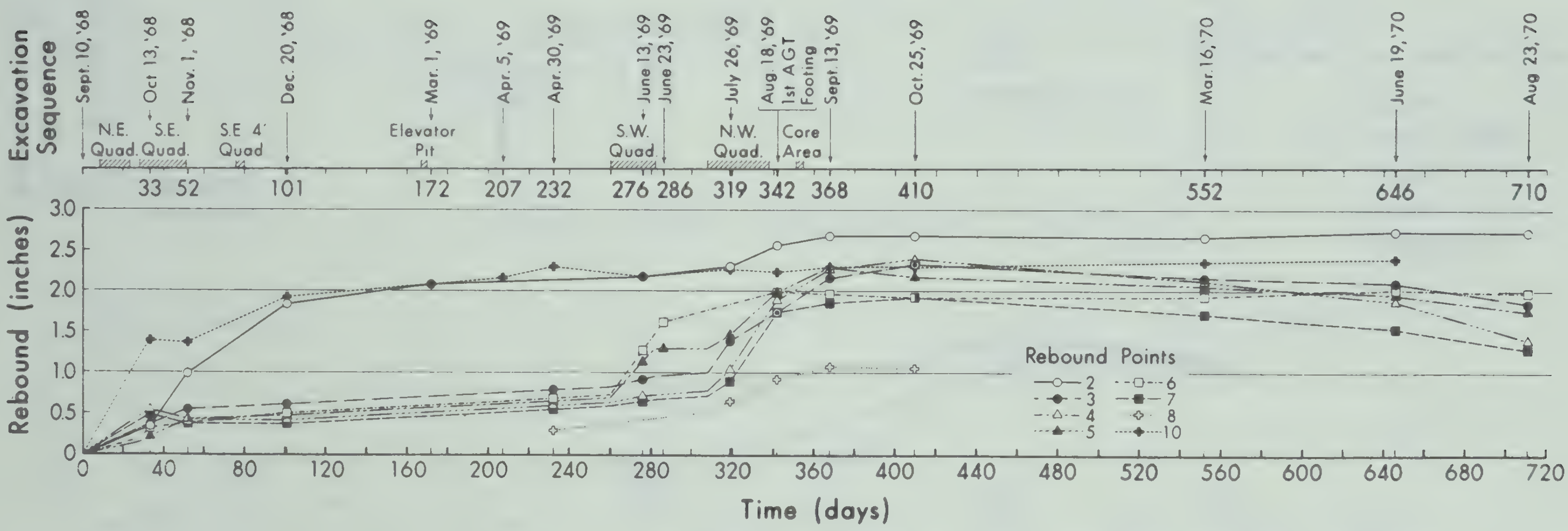

FIG. 5.11 REBOUND HISTORY AT THE AGT EXCAVATION (FROM DEJONG, 1971)

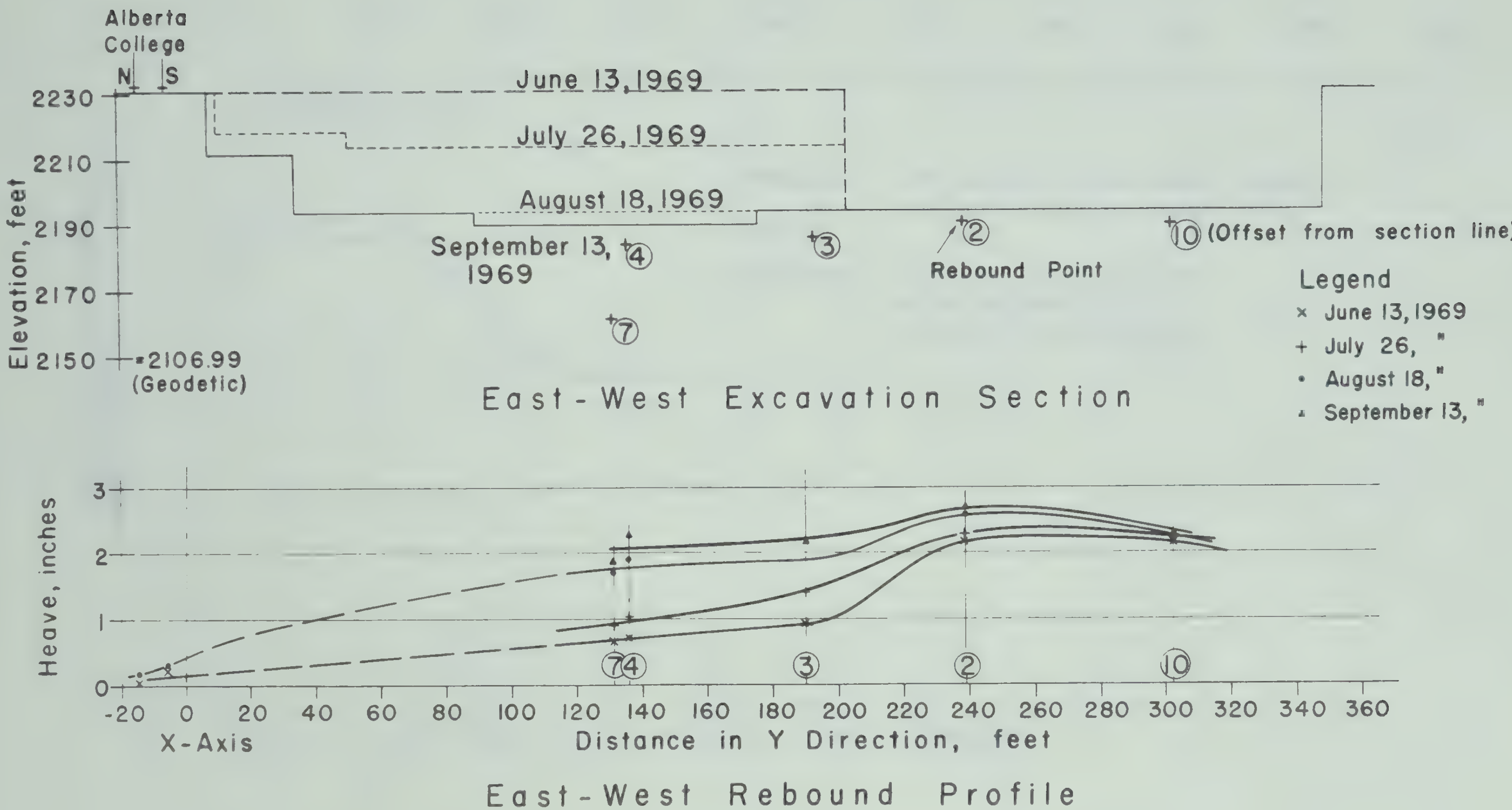

FIG. 5.12 EXCAVATION SECTION AND HEAVE PROFILE - AGT (FROM DEJONG, 1971)

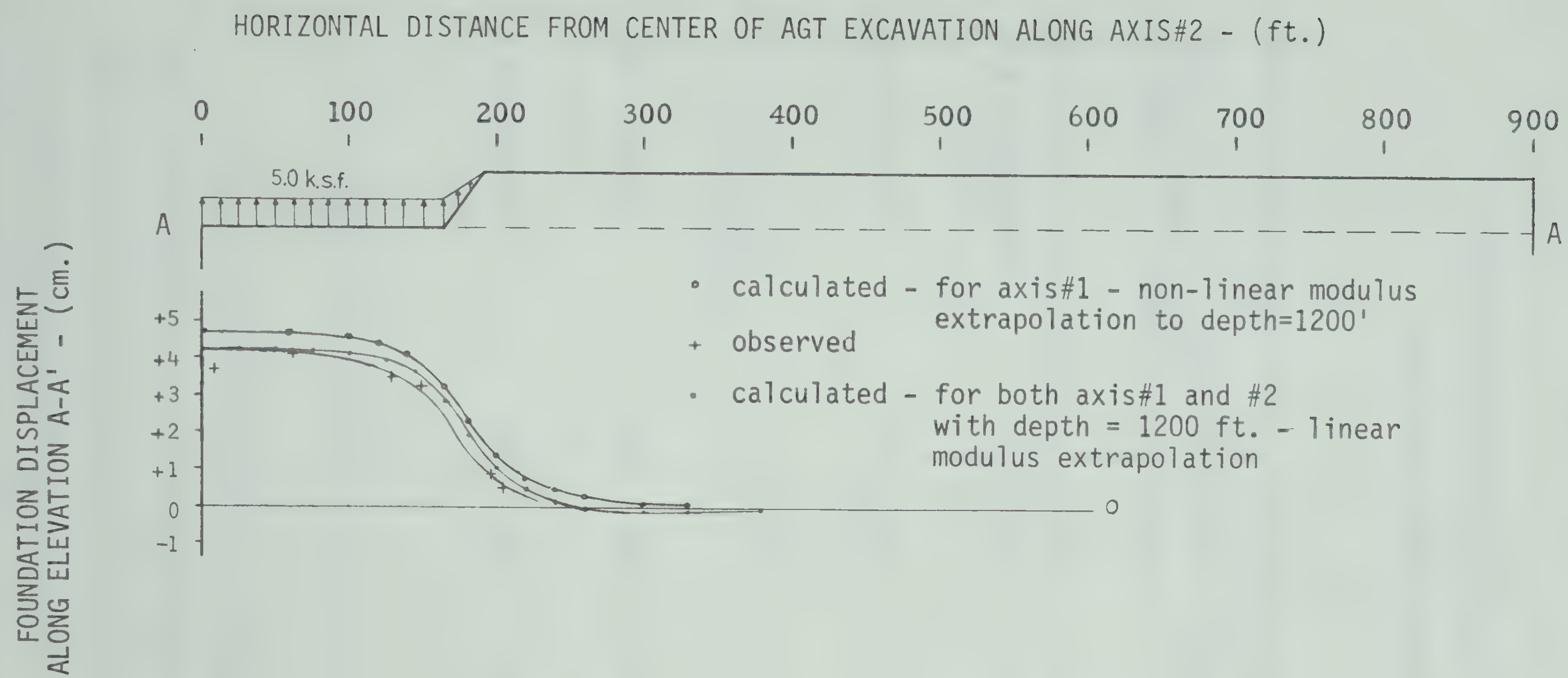

FIG. 5.13 HEAVE DISPLACEMENT PROFILES AT AGT EXCAVATION

TABLE 5.2

ANALYSES PERFORMED AND RESULTS PLOTTED

FOR THE AGT TOWER EXCAVATION

Type of Finite Element Analysis	Plane Strain Axis	Dimensions - ft. Width	Depth	Poisson's Ratio	Results Plotted	Figures Describing Data
2-D (linear mod. extrap.)	2	880	1200	.40	Heave at foundation level	5.13
2-D (linear)	1	900	1200	.40	Heave at foundation level	5.13,5.14
2-D (non-linear mod. extrap.)	1	900	1200	.40	Heave at foundation level	5.13
2-D (linear)	1	900	840	.40	Heave at foundation level	5.14
2-D (linear)	1	900	480	.40	Heave at foundation level	5.14

Fig. 5.13 shows a remarkable correspondence, both in profile and magnitude, of measured and calculated heave. The observed condition of negligible heave outside the excavation area is also demonstrated in the analyses and is generally believed due to increasing deformation modulus with depth (Gibson, 1967 and Ward et al. 1968). The close correspondence of heave magnitude reflects appropriate E values whereas

the similar heave profiles reflect the degree of accuracy of the analysis. A study of Fig. 5.13 also reveals little difference with respect to magnitude and profile of heave between calculations based on the linear extrapolation of E and those based on the non-linear extrapolation of E. It is felt that the linear and non-linear curves used bound the actual condition of modulus variation with depth at the AGT Tower site. Fig. 5.13 would further indicate that the presence of the river bank does not affect the heave magnitude or profile. This is probably due also to increasing stiffness with depth.

One of the problems encountered in settlement or heave analyses is that of deciding on a minimum depth that must be considered for an accurate prediction of deformation. The depth of concern in general will depend primarily upon the areal extent of loading and the non-homogeneity of the foundation material, particularly in regard to deformation properties. If the foundation is assumed to be homogeneous, calculated deformations do not reach an early asymptote with respect to depth of assumed rigid base (Matheson, 1972). However under non-homogeneous foundation conditions, in which E increases linearly with depth, at the rate indicated in Fig. 4.4, calculated deformations reach an asumptote relatively soon with respect to depth of assumed rigid base (Fig. 5.13).

Based on the linear extrapolation of E as shown in Fig. 4.4, it would have been sufficient to consider a depth of about 1.5 times the width of the loaded area for the AGT excavation heave analysis. It should be recognized however that a lesser modulus gradient such as the non-linear one considered in Fig. 4.4 would require a greater depth of concern in the deformation analysis.

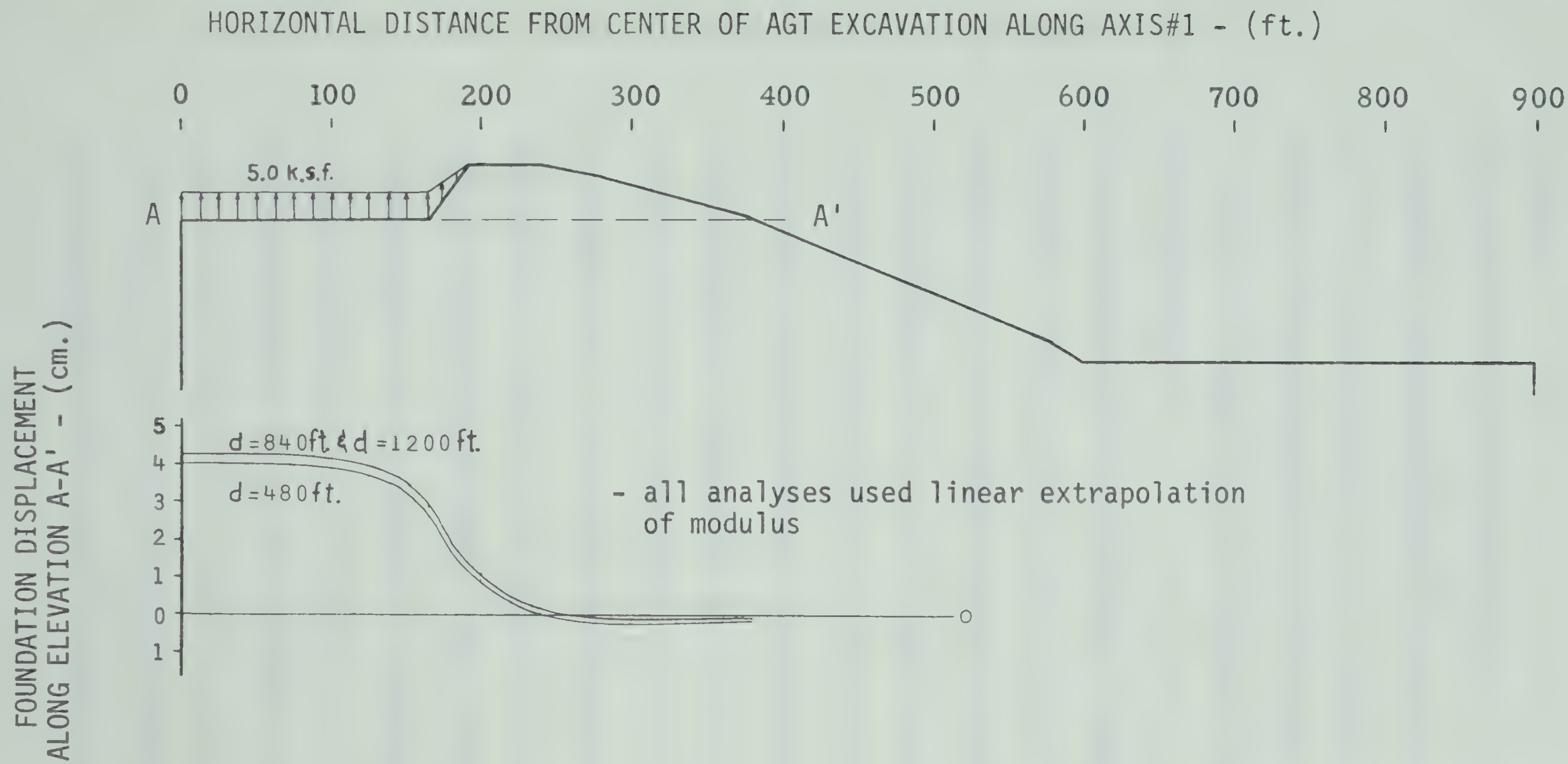

FIG. 5.14 HEAVE DISPLACEMENT PROFILES AT AGT EXCAVATION AS A FUNCTION OF DEPTH OF FINITE ELEMENT GRID

5.5 Conclusions

1. The pressuremeter deformation moduli in conjunction with a rationally selected Poisson's ratio of 0.42, when used in a finite element analysis, will accurately predict the elastic heave in open excavations and the elastic settlement of structures situated on the overconsolidated soils of downtown Edmonton.
2. A two-dimensional constant strain triangle finite element analysis in plane strain can be used with the same degree of accuracy as a three-dimensional isoparametric hexahedron finite element analysis for predicting maximum total and differential settlement of a uniform flexible loading of area 158 x 110 square feet. This does not invalidate the two-dimensional analysis if the loaded area is less than that of the CN tower foundation.
3. Two-dimensional finite element analyses are sensitive to the value of Poisson's ratio and therefore its value should not be arbitrarily selected. The selection should be a rational one based on degree of saturation, permeability, structure and density of the soil.
4. Soil foundations exhibiting increasing modulus with depth, give rise to deformation that are confined essentially to the loaded area and the calculated deformations reach an early asymptote with respect to depth of assumed rigid base.

CHAPTER VI

CONCLUSIONS

The main objective of this thesis was to investigate the reliability of the pressuremeter as a device for obtaining representative in situ moduli that could be used in an elastic analyses for the prediction of immediate settlement or heave. A limited amount of laboratory testing was performed to demonstrate the degree of correspondence between moduli obtained from the pressuremeter test and those obtained from traditional laboratory testing.

The results of this technical research are listed below.

1. The moduli obtained from the pressuremeter tests, used in a finite element analysis in conjunction with a rationally selected Poisson's ratio will accurately predict elastic deformations of geotechnical concern in the overconsolidated soils and bedrock in Central Edmonton.
2. The magnitude of the moduli obtained with the pressuremeter was significantly greater than the comparable laboratory values obtained in this study and by DeJong (1971) in his investigation at the same locations. The difference approached an order of magnitude.
3. Limited laboratory results indicate that deformation modulus is sensitive to the stress path taken to failure. However it appears that regardless of the stress path taken, the initial tangent modulus will be low by a factor of from 5 to 7.
4. A two-dimensional constant strain triangle finite element

analysis in plane strain was used with the same degree of accuracy as a three-dimensional isoparametric hexahedron finite element analysis in predicting maximum total and differential settlement of a uniform flexible loading of area 158 by 110 square feet. The average computer cost of the two-dimensional analyses was \$4.50 and \$550.00 for the three-dimensional analyses.

5. Two-dimensional finite element analyses are sensitive to the value of Poisson's ratio (μ). Arbitrary selection of μ is therefore discouraged in favor of a rationally obtained value in which such conditions as degree of saturation, permeability, structure and density of the soil are considered.
6. Upon encountering intact bedrock, the moduli obtained from the pressuremeter tests showed a definite dependence on confining pressure. The deformation modulus increased with depth in approximately the same manner for the two sites (CN and AGT Towers) investigated. An insufficient number of pressuremeter tests were performed to accurately establish the actual mathematical relationship between deformation modulus and depth, therefore both a linear and power law relationship were considered.
7. Soil foundations exhibiting increasing modulus with depth, give rise to deformations that are confined essentially to the loaded area and calculated deformations reach an early asymptote with respect to depth of assumed rigid base.
8. Pressuremeter tests in the fractured shale at the AGT tower site indicated the anticipated macrostructural control of the deform-

ation modulus. The volume-time curves exhibited elastic-plastic like behavior at low stress levels which can readily be explained in terms of deformation response to changes in applied stress resulting in the closing of fissures.

9. In the pressuremeter testing of dense soils that have a propensity to become disturbed, if the deposit is known to be relatively homogeneous, then it may be acceptable policy to regard only the highest measured E values as being representative of the in situ stiffness of the soil. However, attention must be given to the possibility of increasing modulus with depth and to the acceptability of the volume-time curves from which the respective moduli were calculated, before the decision of acceptance or rejection of moduli is made.
10. There appears to be little need in extending volume-time measurements beyond the elastic time interval (Te) for each pressure increment. However many readings should be taken within this interval.
11. The pressure probe is very sensitive to borehole diameter, requiring close tolerance for effective performance. This condition is the result of lack of continuous restriction of longitudinal expansion of the measuring cell with increased borehole diameter. Expandable washers situated at either end of the measuring cell is suggested as a possible design solution.
12. There is a need for a system of water control valves to be situated within the probe to allow pressuremeter testing in dry boreholes at depths greater than about 25 feet.
13. There is an economic need for a 4 inch diameter probe from

the standpoint of existing drilling facilities in the Edmonton area.

14. The cost per pressuremeter test in this study was $110 which does not include the pressuremeter operator's wage. At least a 50% reduction in cost should be attainable through non-restricted drilling procedures required upon implementing probe design improvements. The cost per Pitcher sample retrieved was $100 from which a projected cost of about $130 per modulus valve was estimated. This cost is conservative in that all samples retrieved can not be used for testing. This difference in cost along with the low degree of confidence associated with the deformation modulus obtained from conventional laboratory tests makes an investigation employing pressuremeter testing attractive both technically and economically.

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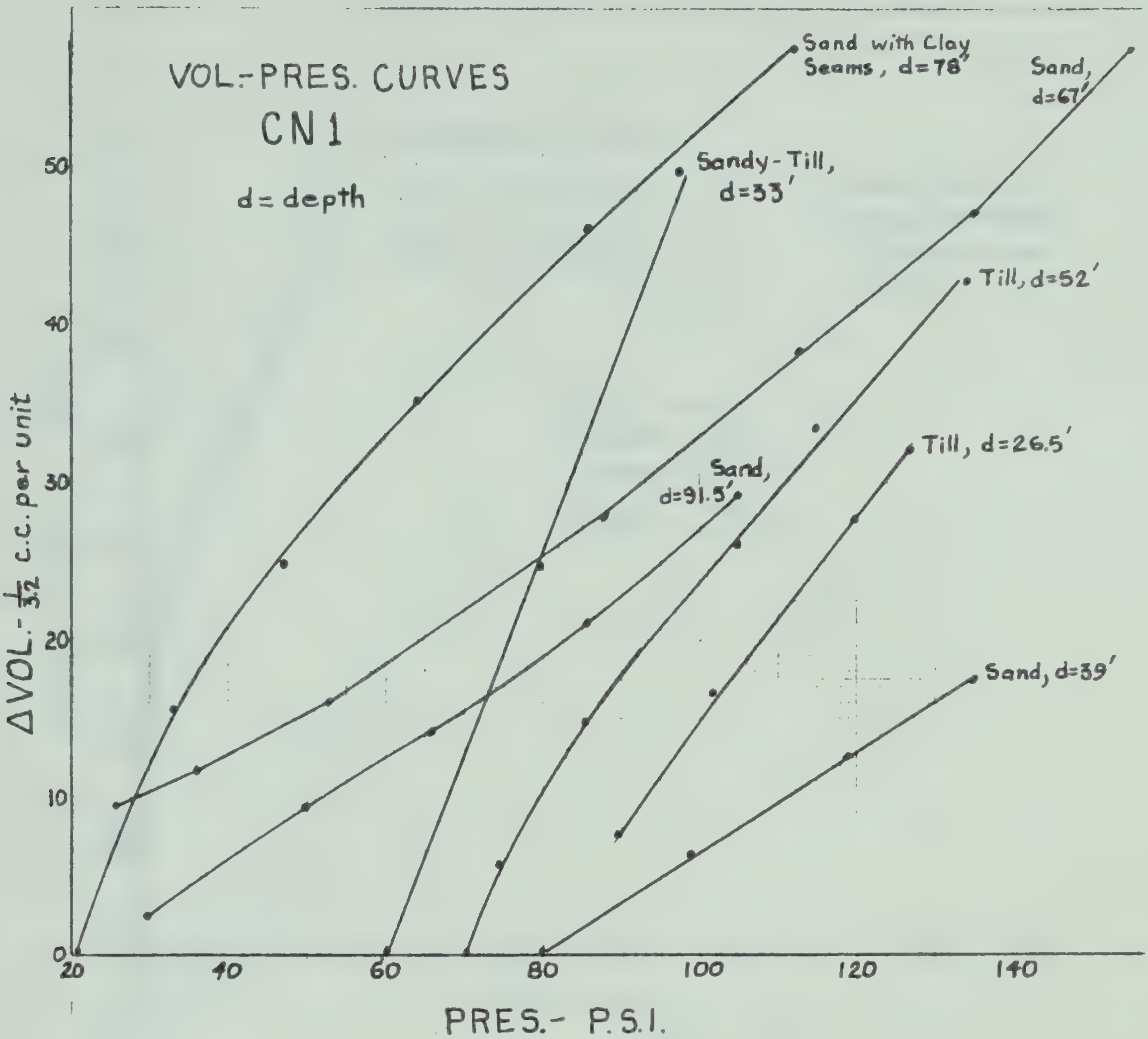
APPENDIX A

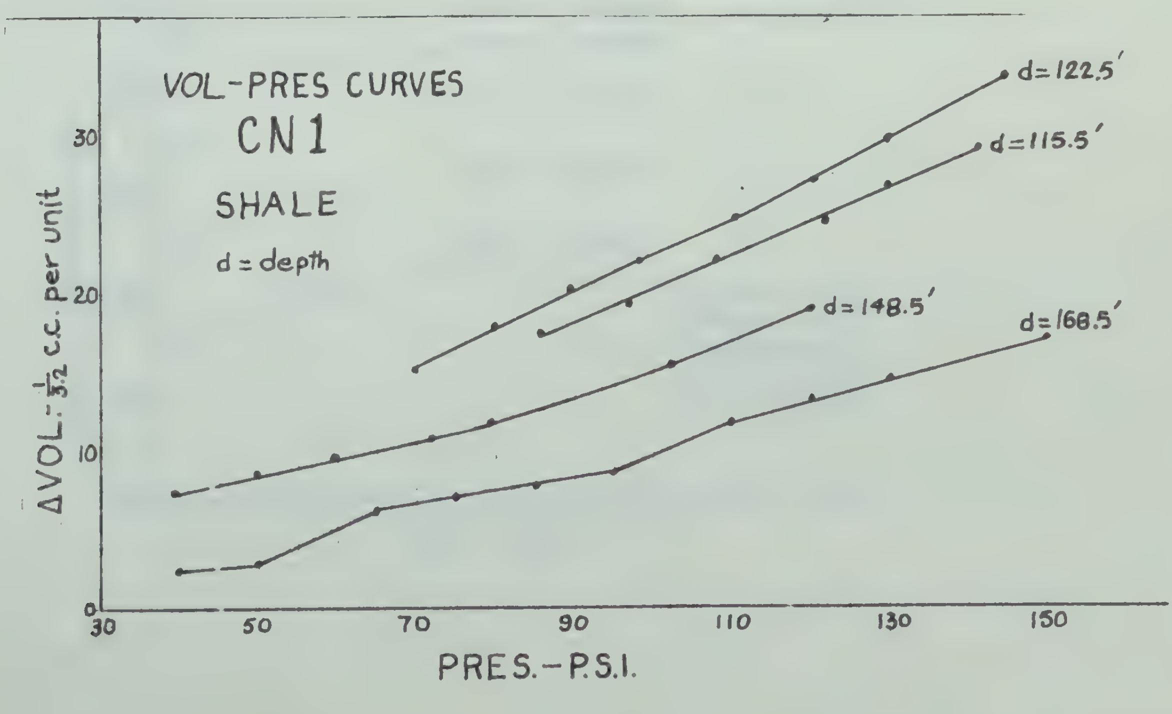
PRESSUREMETER

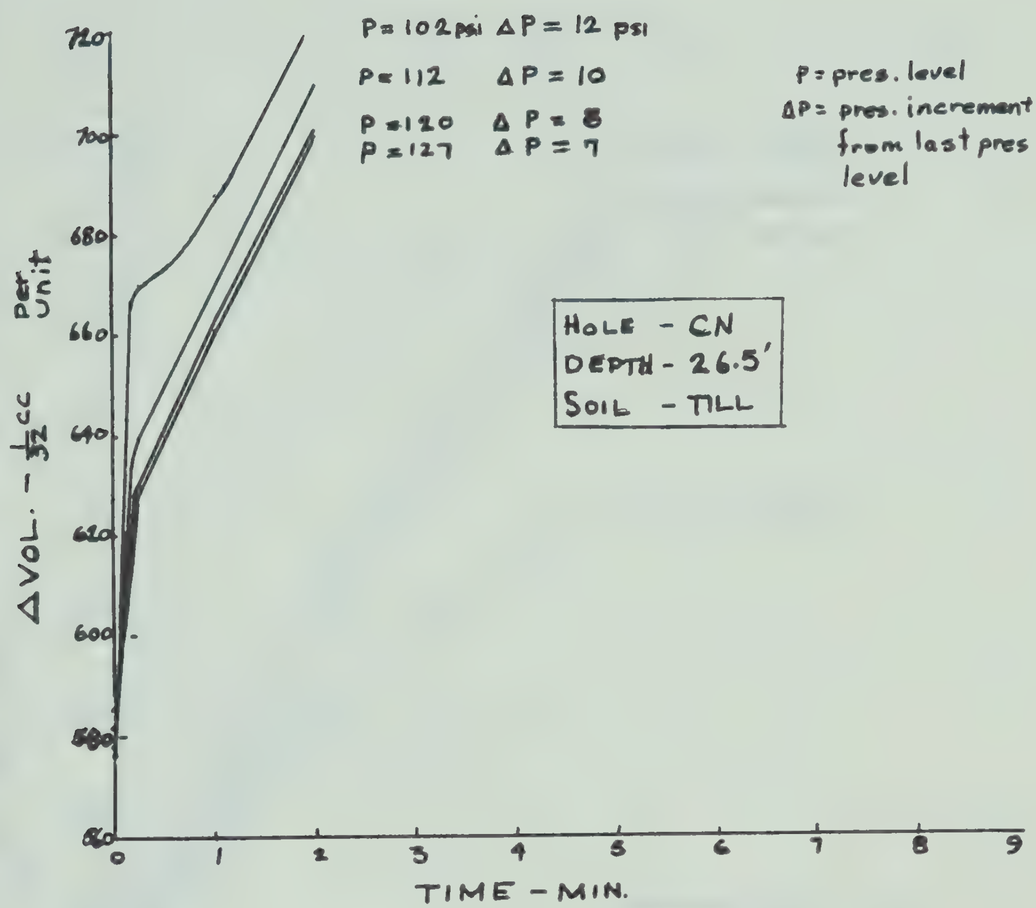
VOLUME-TIME AND

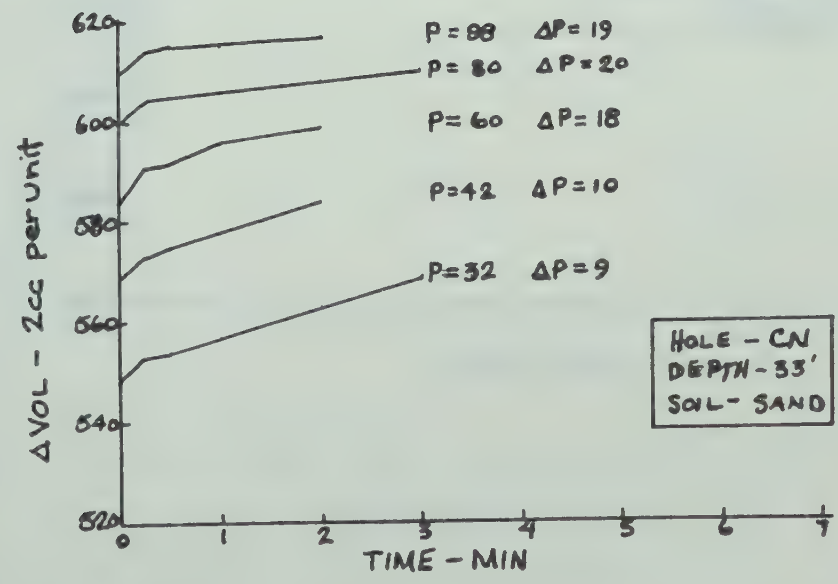
VOLUME-PRESSURE CURVES

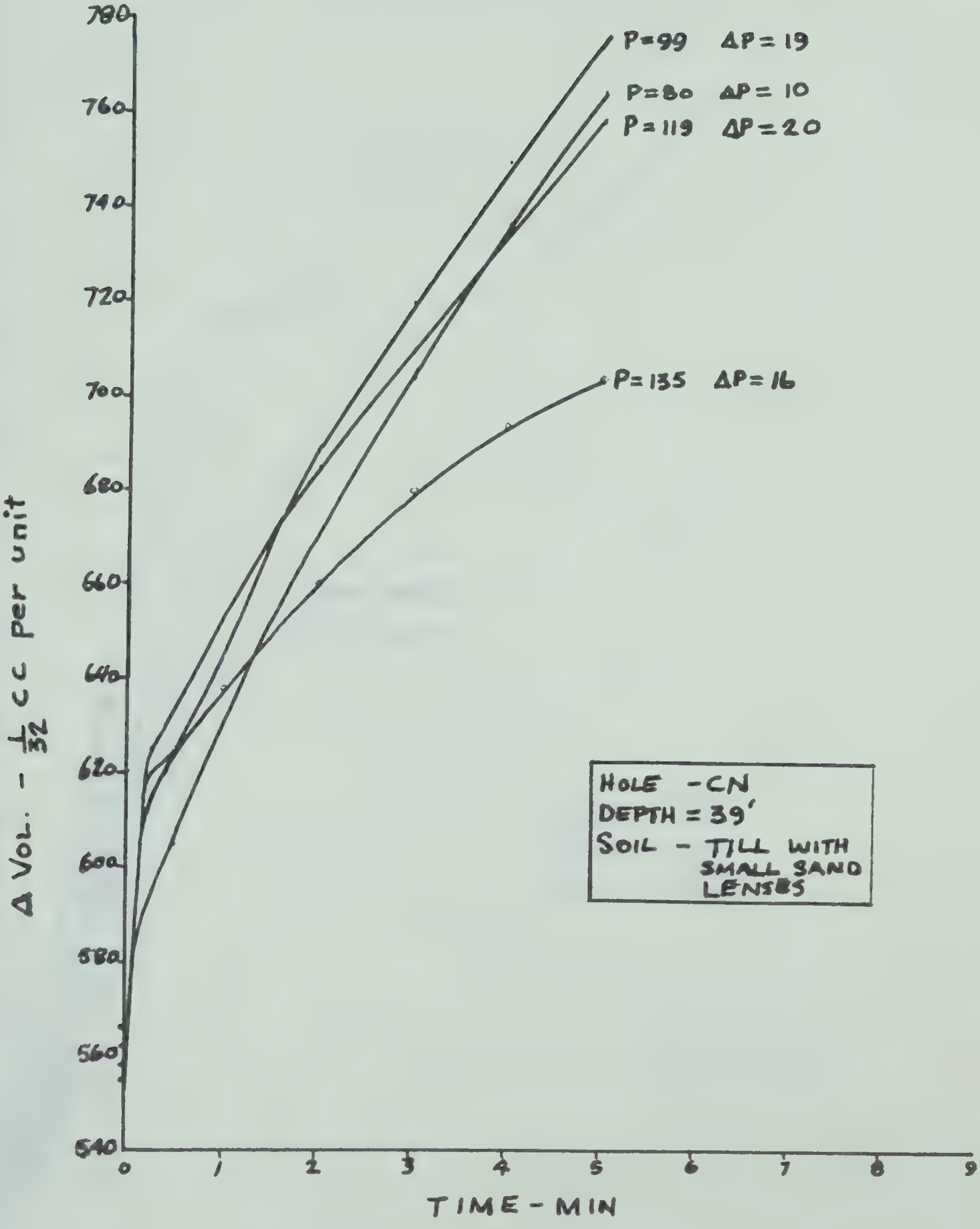
- CN TOWER -

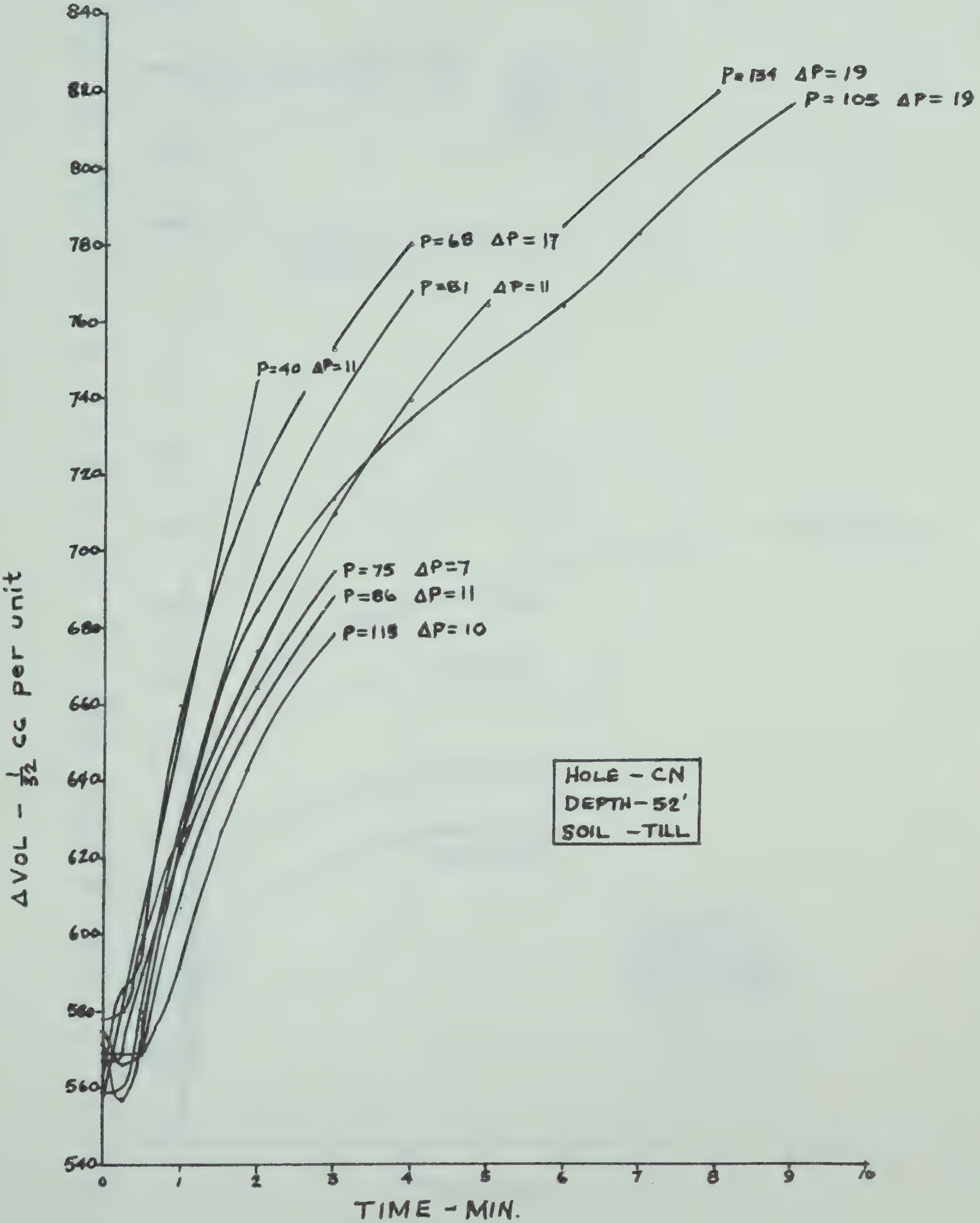

VOL.-PRES. CURVES
CN1
d = depth
Sand with Clay Seams, d=78'
Sand, d=67'
Sandy-Till, d=33'
Till, d=52'
Till, d=26.5'
Sand, d=91.5'
Sand, d=39'
ΔVOL.- 1/3.2 c.c. per unit
PRES.- P.S.I.
0
10
20
30
40
50
20
40
60
80
100
120
140

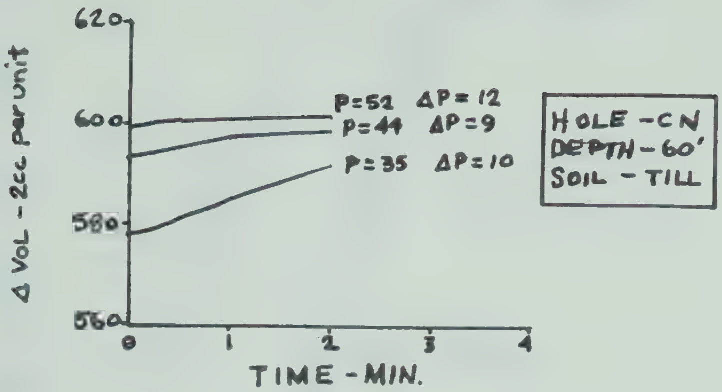

VOL.-PRES CURVES
CN1
SHALE
d = depth
d=122.5'
d=115.5'
d=148.5'
d=168.5'
ΔVOL.- 1/3.2 c.c. per unit
PRES.-P.S.I.
0
10
20
30
30
50
70
90
110
130
150

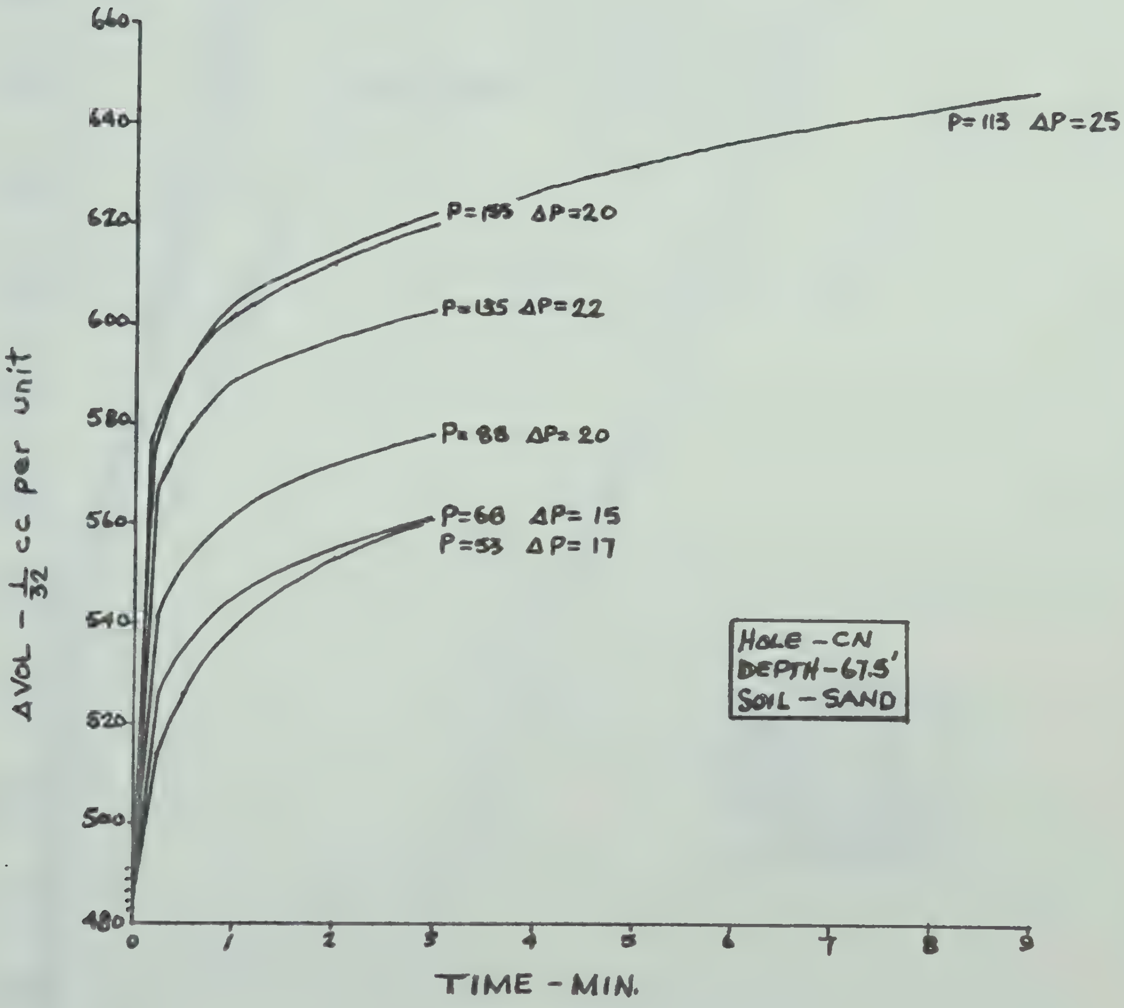

P= 102 psi ΔP = 12 psi
P= 112 ΔP = 10
P= 120 ΔP = 8
P= 127 ΔP = 7
P = pres. level
ΔP = pres. increment from last pres level
HOLE - CN
DEPTH - 26.5'
SOIL - TILL
ΔVOL. - 1/32 cc per unit
TIME - MIN.

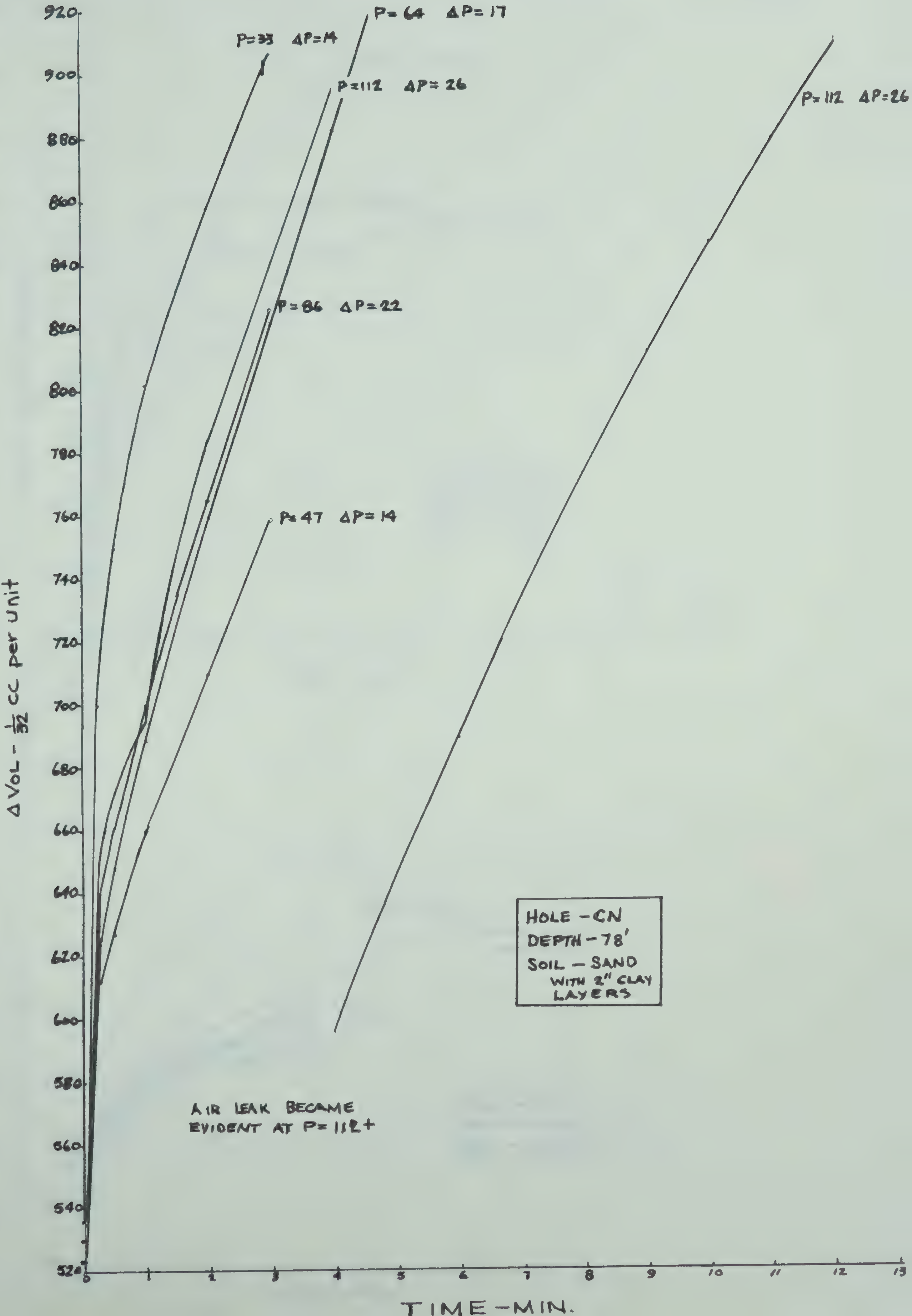

P = 98 ΔP = 19
P = 80 ΔP = 20
P = 60 ΔP = 18
P = 42 ΔP = 10
P = 32 ΔP = 9
HOLE - CN
DEPTH - 33'
SOIL - SAND
ΔVOL - 2cc per unit
TIME - MIN

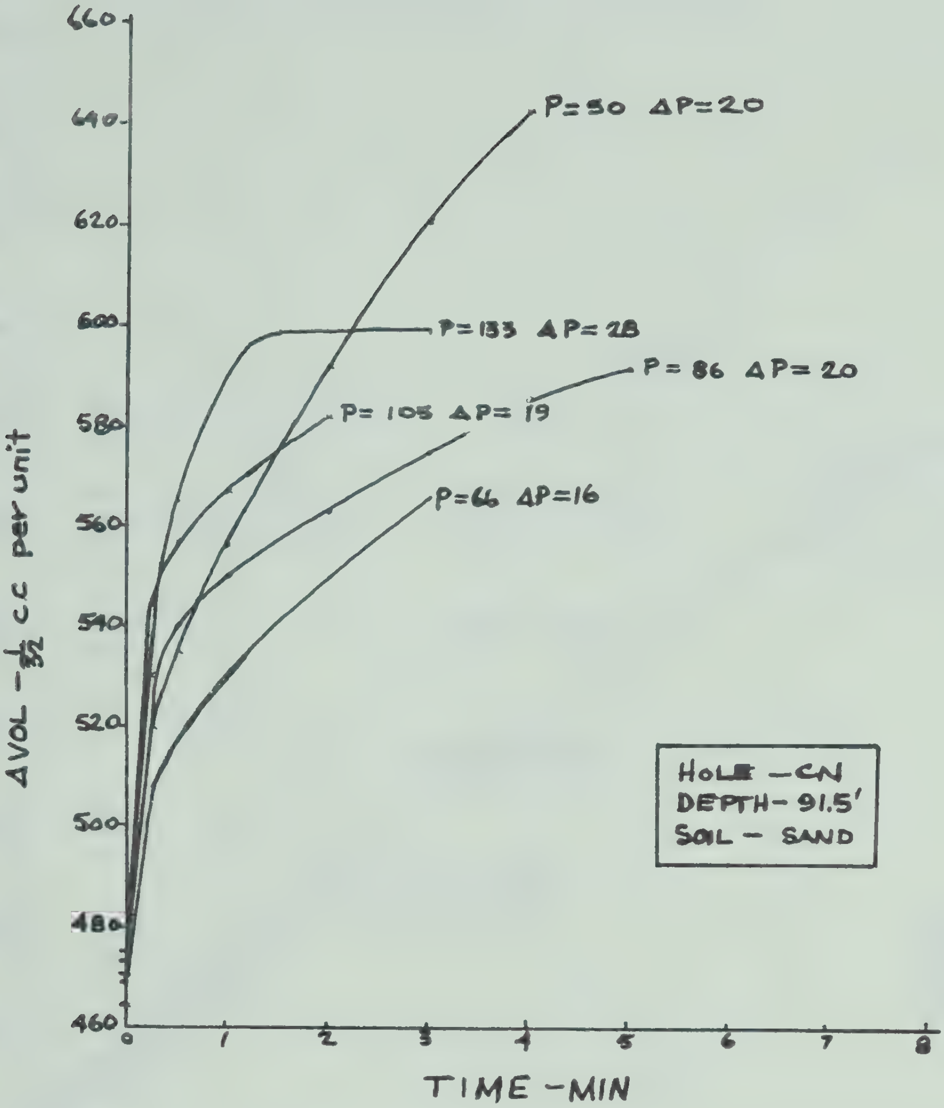

P=99 ΔP=19
P=80 ΔP=10
P=119 ΔP=20
P=135 ΔP=16
HOLE -CN
DEPTH = 39'
SOIL - TILL WITH SMALL SAND LENSES
Δ VOL. - 1/32 cc per unit
TIME - MIN
780
760
740
720
700
680
660
640
620
600
580
560
540
0
1
2
3
4
5
6
7
8
9

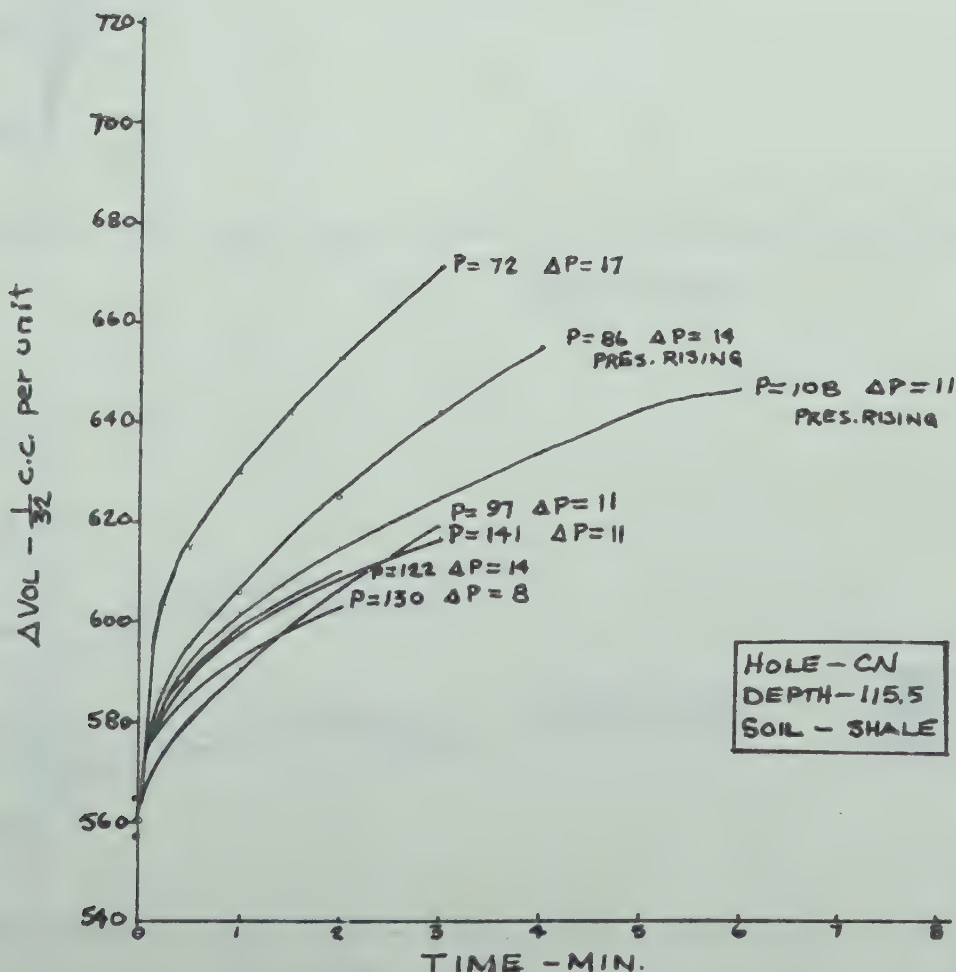

P=134 ΔP=19
P=105 ΔP=19
P=68 ΔP=17
P=81 ΔP=11
P=40 ΔP=11
P=75 ΔP=7
P=86 ΔP=11
P=115 ΔP=10
HOLE - CN
DEPTH - 52'
SOIL - TILL
ΔVOL - 1/32 cc per unit
TIME - MIN.
840
820
800
780
760
740
720
700
680
660
640
620
600
580
560
540
0
1
2
3
4
5
6
7
8
9
10

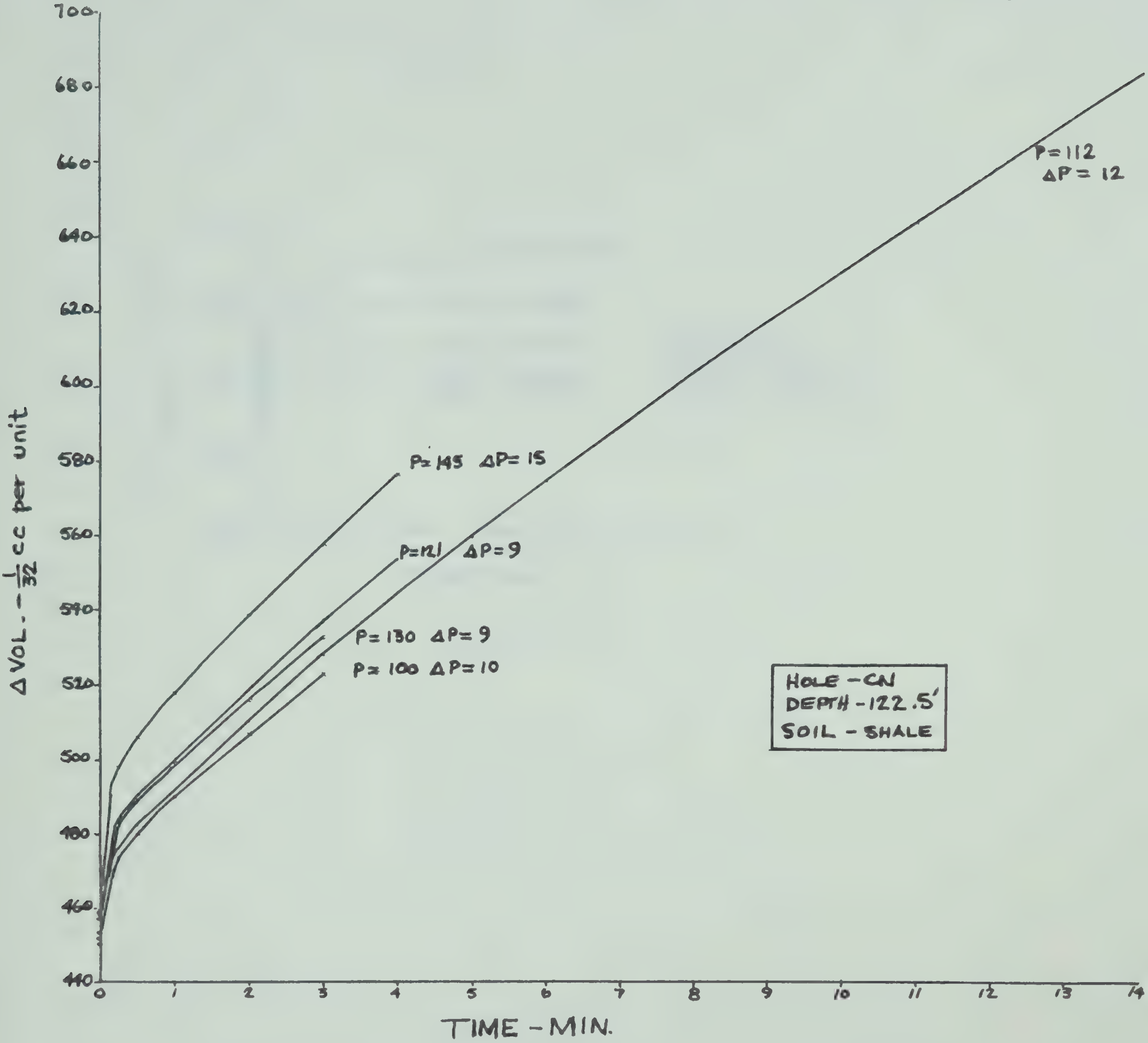

620
600
580
560
Δ VOL - 2cc per unit
0
1
2
3
4
TIME - MIN.
P=52 ΔP=12
P=44 ΔP=9
P=35 ΔP=10
HOLE - CN
DEPTH - 60'
SOIL - TILL


660
640
620
600
580
560
540
520
500
480
ΔVOL - 1/32 cc per unit
0
1
2
3
4
5
6
7
8
9
TIME - MIN.
P=113 ΔP=25
P=155 ΔP=20
P=135 ΔP=22
P=88 ΔP=20
P=68 ΔP=15
P=53 ΔP=17
HOLE - CN
DEPTH - 67.5'
SOIL - SAND

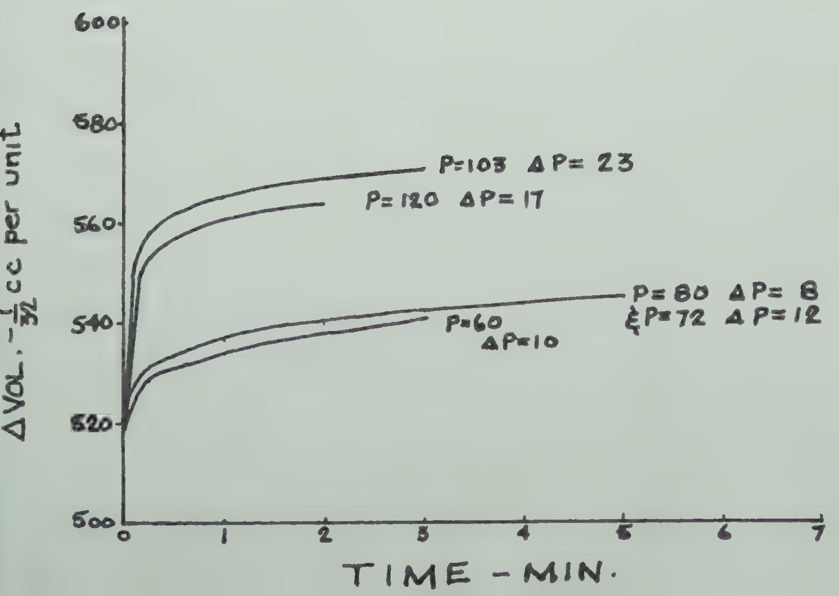

P=33 ΔP=14
P=64 ΔP=17
P=112 ΔP=26
P=112 ΔP=26
P=86 ΔP=22
P=47 ΔP=14
HOLE - CN
DEPTH - 78'
SOIL - SAND WITH 2" CLAY LAYERS
AIR LEAK BECAME EVIDENT AT P=112+
ΔVOL - 1/32 CC per unit
TIME - MIN.
920
900
880
860
840
820
800
780
760
740
720
700
680
660
640
620
600
580
560
540
520
0
1
2
3
4
5
6
7
8
9
10
11
12
13


P=50 ΔP=20
P=133 ΔP=28
P=86 ΔP=20
P=105 ΔP=19
P=66 ΔP=16
ΔVOL - 1/32 c.c per unit
TIME - MIN
HOLE - CN
DEPTH - 91.5'
SOIL - SAND

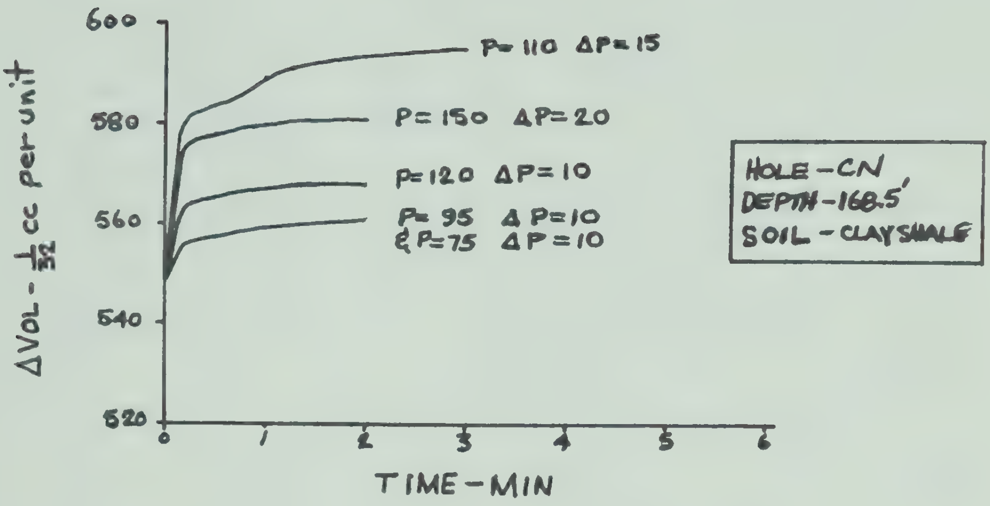

P=72 ΔP=17
P=86 ΔP=14
PRES. RISING
P=108 ΔP=11
PRES. RISING
P=97 ΔP=11
P=141 ΔP=11
P=122 ΔP=14
P=130 ΔP=8
ΔVOL - 1/32 c.c. per unit
TIME - MIN.
HOLE - CN
DEPTH - 115.5
SOIL - SHALE



700
680
660
640
620
600
580
560
540
520
500
480
460
440
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14
ΔVOL. - 1/32 cc per unit
TIME - MIN.
P=112
ΔP = 12
P=145 ΔP= 15
P=121 ΔP=9
P= 130 ΔP= 9
P= 100 ΔP=10
HOLE - CN
DEPTH - 122.5'
SOIL - SHALE



600
580
560
540
520
500
0 1 2 3 4 5 6 7
ΔVOL. - 1/32 cc per unit
TIME - MIN.
P=103 ΔP= 23
P= 120 ΔP= 17
P= 80 ΔP= 8
& P= 72 ΔP= 12
P=60
ΔP=10
HOLE - CN
DEPTH - 148.5
SOIL - SHALE


600
580
560
540
520
ΔVOL - 1/32 cc per unit
P= 110 ΔP= 15
P= 150 ΔP= 20
P= 120 ΔP= 10
P= 95 ΔP=10
& P=75 ΔP=10
0
1
2
3
4
5
6
TIME-MIN
HOLE-CN,
DEPTH-168.5'
SOIL-CLAYSHALE

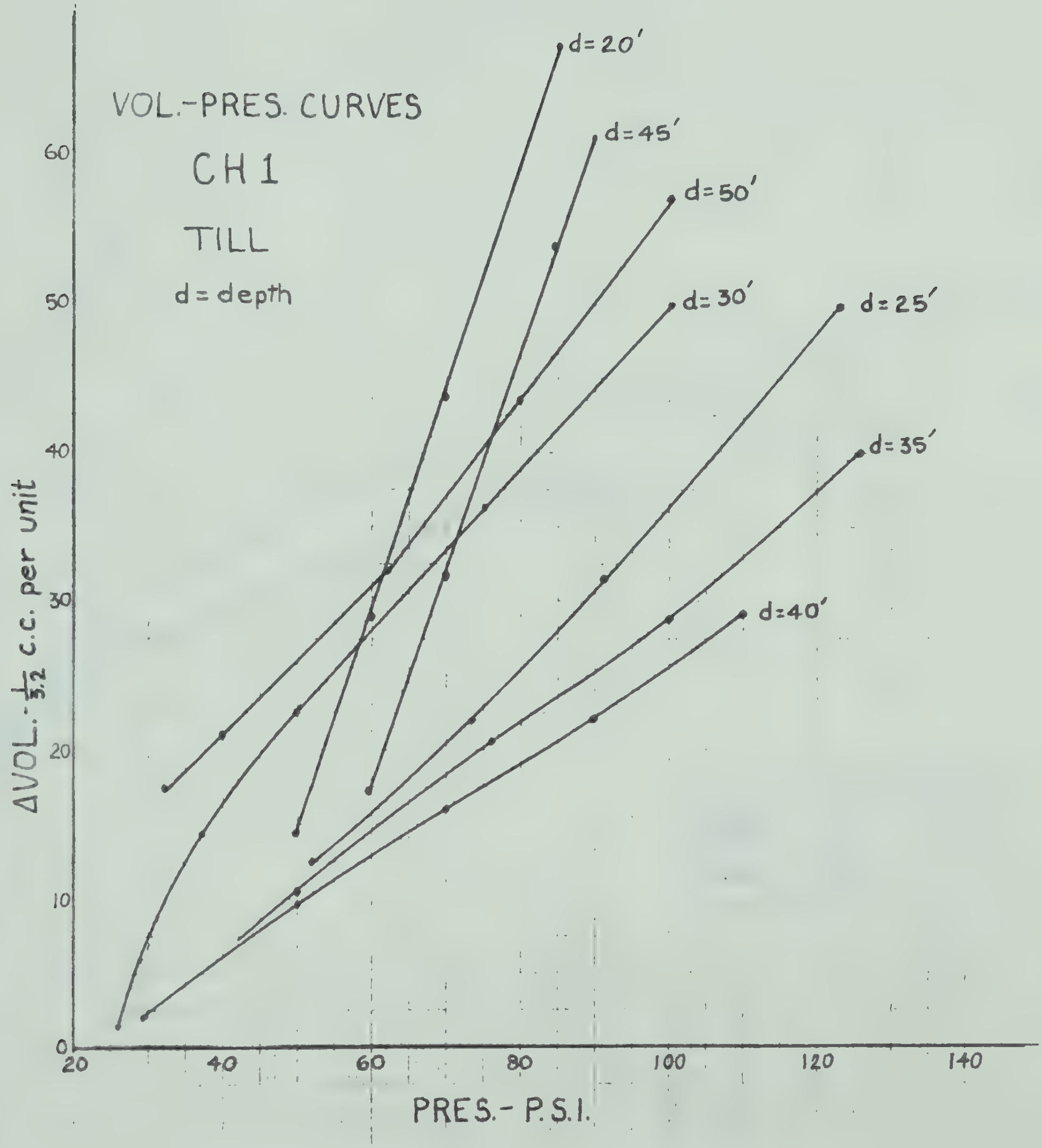
APPENDIX B

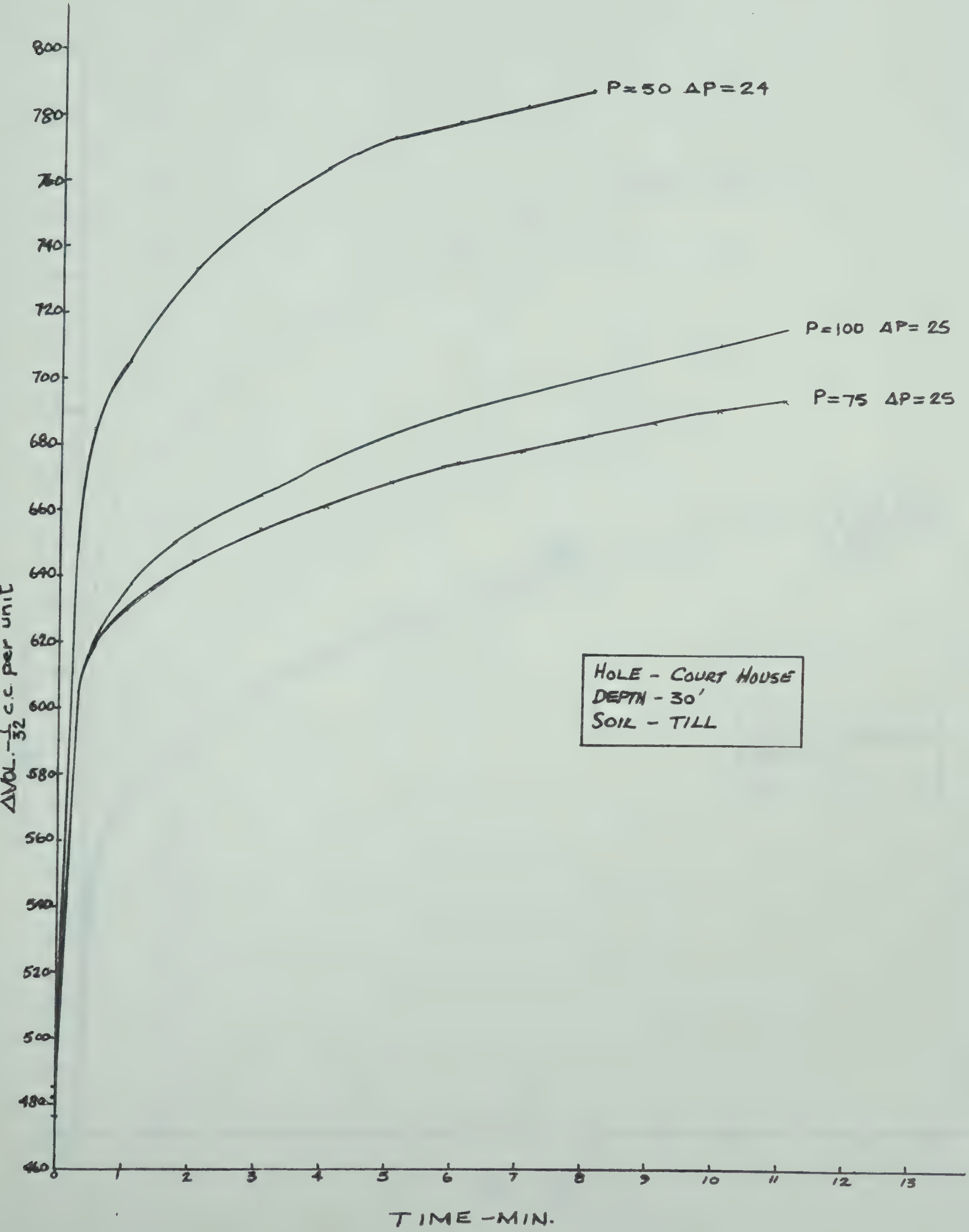
PRESSUREMETER

VOLUME-TIME AND

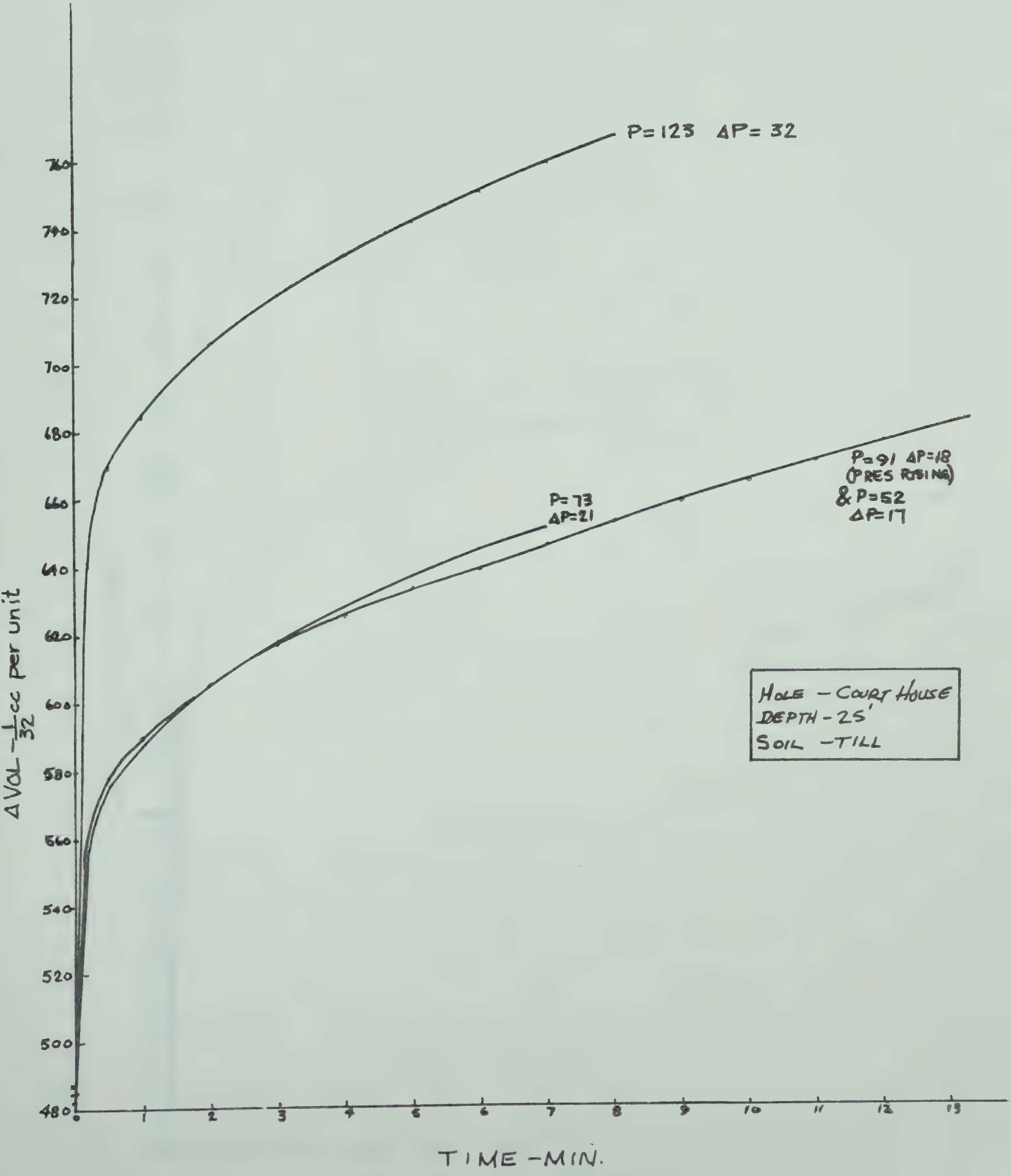
VOLUME-PRESSURE CURVES

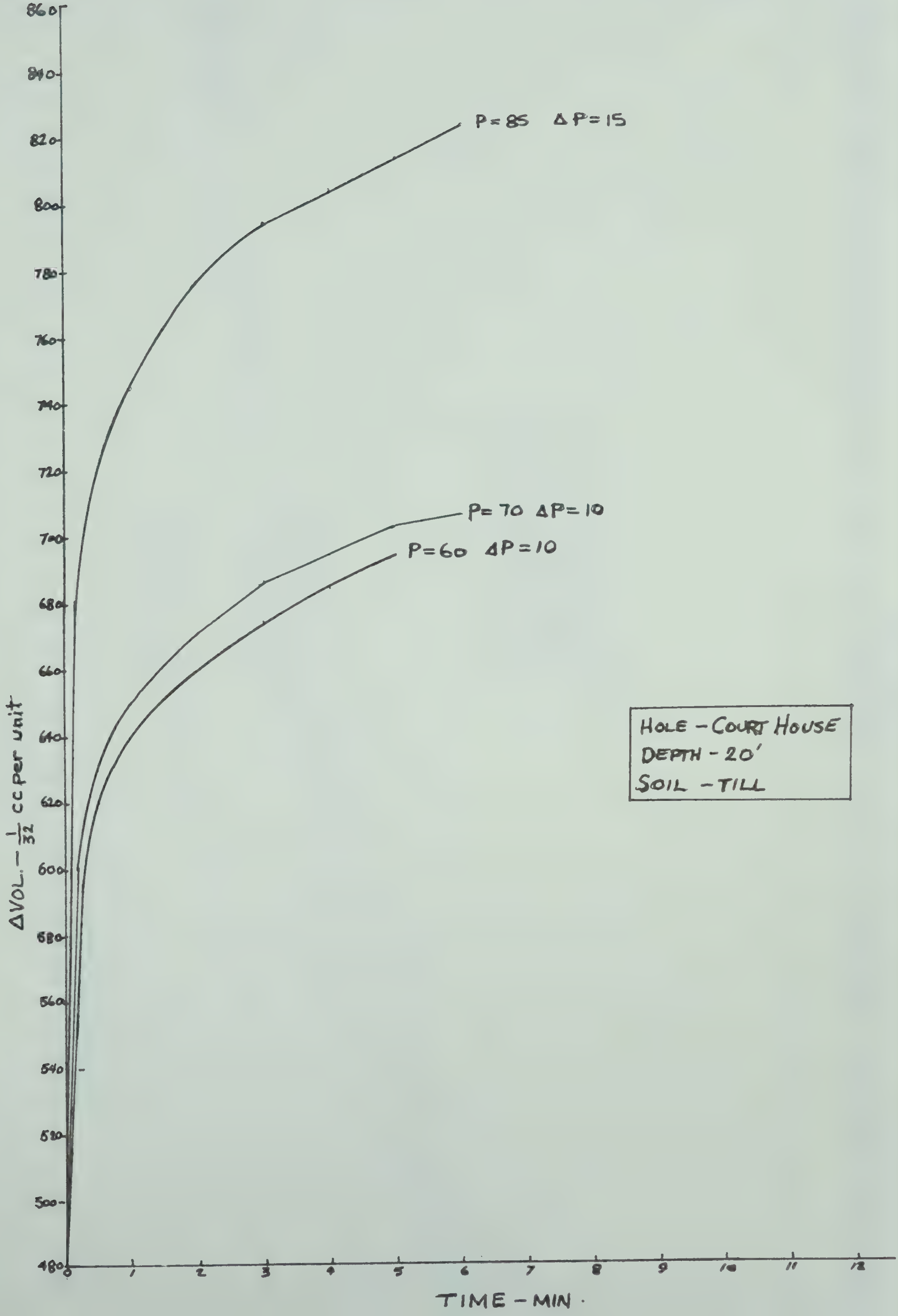
- COURT HOUSE -

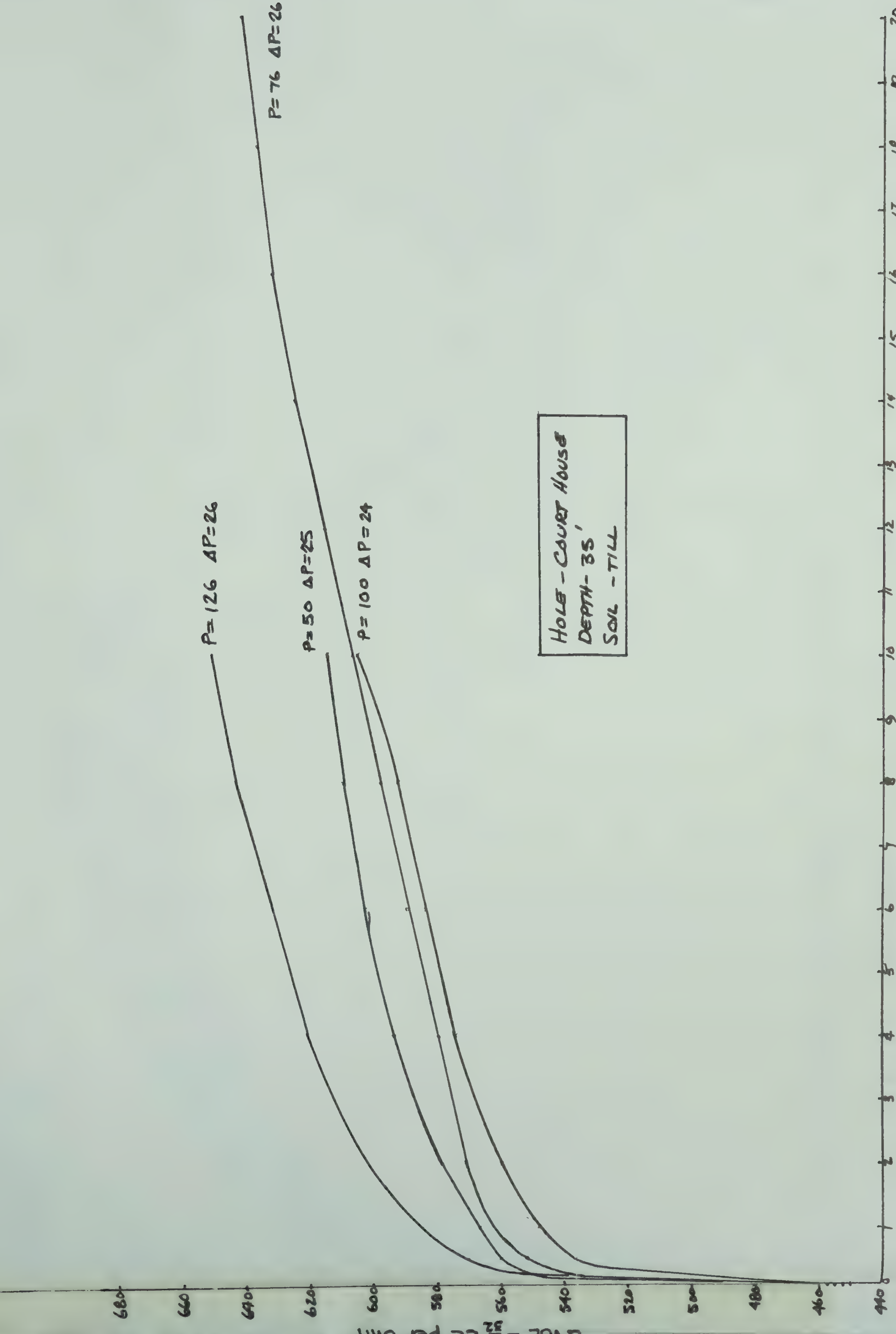

VOL.-PRES. CURVES
CH 1
TILL
d = depth
d=20′
d=45′
d=50′
d=30′
d=25′
d=35′
d=40′
ΔVOL.·1/3.2 c.c. per unit
PRES.- P.S.I.
0
10
20
30
40
50
60
20
40
60
80
100
120
140



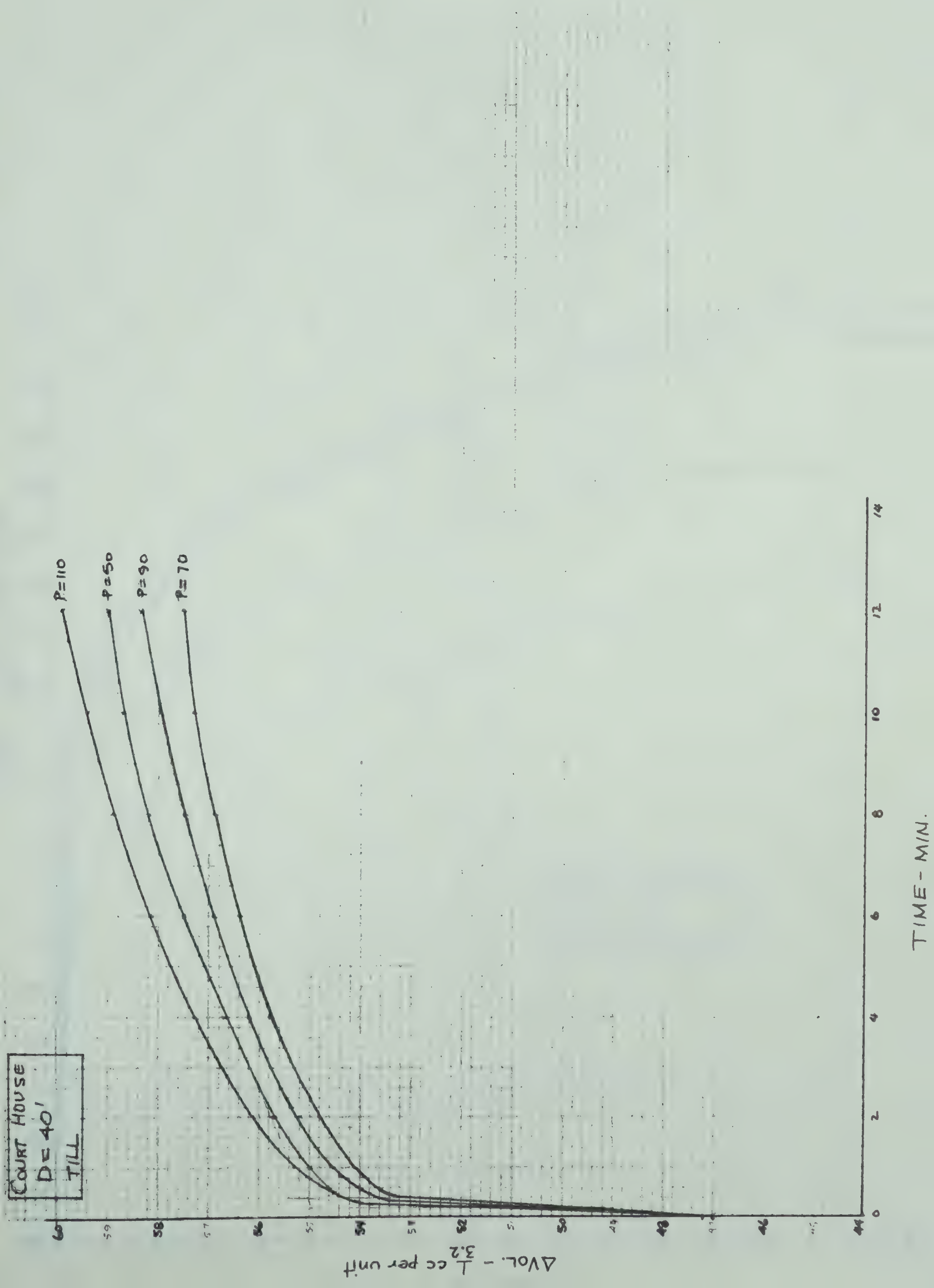
P=50 ΔP=24
P=100 ΔP=25
P=75 ΔP=25
HOLE - COURT HOUSE
DEPTH - 30'
SOIL - TILL
ΔVOL. - 1/32 c.c. per unit
TIME - MIN.
800
780
760
740
720
700
680
660
640
620
600
580
560
540
520
500
480
460
0
1
2
3
4
5
6
7
8
9
10
11
12
13

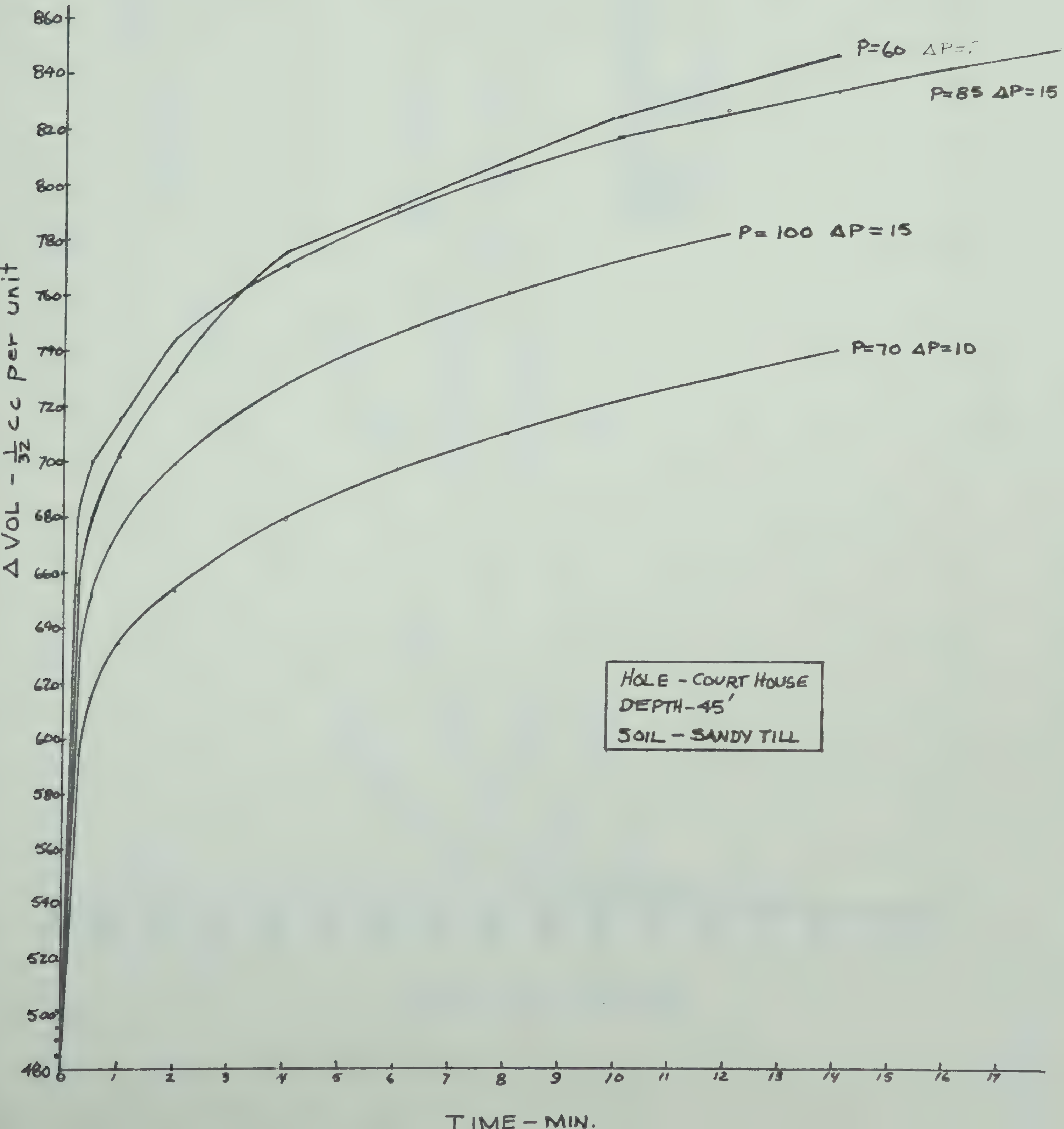

P=123 ΔP=32
P=73
ΔP=21
P=91 ΔP=18
(PRES RISING)
& P=52
ΔP=17
HOLE - COURT HOUSE
DEPTH - 25'
SOIL - TILL
ΔVOL - 1/32 cc per unit
TIME - MIN.
760
740
720
700
680
660
640
620
600
580
560
540
520
500
480
0
1
2
3
4
5
6
7
8
9
10
11
12
13

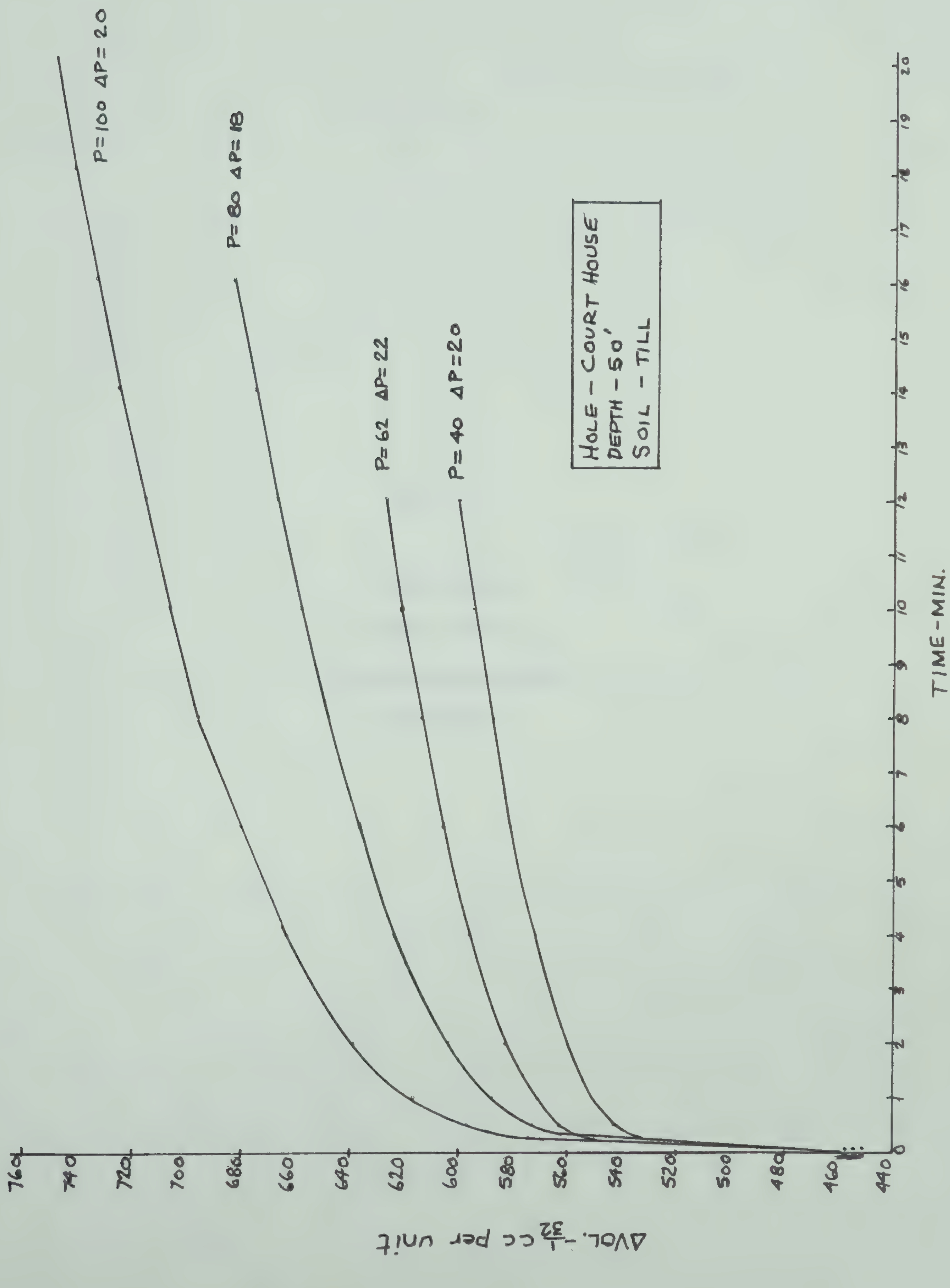

P=85 ΔP=15
P=70 ΔP=10
P=60 ΔP=10
HOLE - COURT HOUSE
DEPTH - 20'
SOIL - TILL
ΔVOL. - 1/32 cc per unit
TIME - MIN.
860
840
820
800
780
760
740
720
700
680
660
640
620
600
580
560
540
520
500
480
0
1
2
3
4
5
6
7
8
9
10
11
12



P=76 ΔP=26
P=126 ΔP=26
P=50 ΔP=25
P=100 ΔP=24
HOLE - COURT HOUSE
DEPTH - 35'
SOIL - TILL
TIME - MIN
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
440
460
480
500
520
540
560
580
600
620
640
660
680


COURT HOUSE
D=40'
TILL
P=110
P=50
P=90
P=70
TIME - MIN.
ΔVOL. - 1/3.2 cc per unit
0
2
4
6
8
10
12
14
44
46
48
50
52
54
56
58
60


P=60 ΔP=5
P=85 ΔP=15
P=100 ΔP=15
P=70 ΔP=10
HOLE - COURT HOUSE
DEPTH - 45'
SOIL - SANDY TILL
Δ VOL - 1/32 cc per unit
TIME - MIN.
860
840
820
800
780
760
740
720
700
680
660
640
620
600
580
560
540
520
500
480
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17


P=100 ΔP=20
P=80 ΔP=18
P=62 ΔP=22
P=40 ΔP=20
HOLE - COURT HOUSE
DEPTH - 50'
SOIL - TILL
TIME-MIN.
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
ΔVOL. - 1/32 cc per unit
440
460
480
500
520
540
560
580
600
620
640
660
680
700
720
740
760

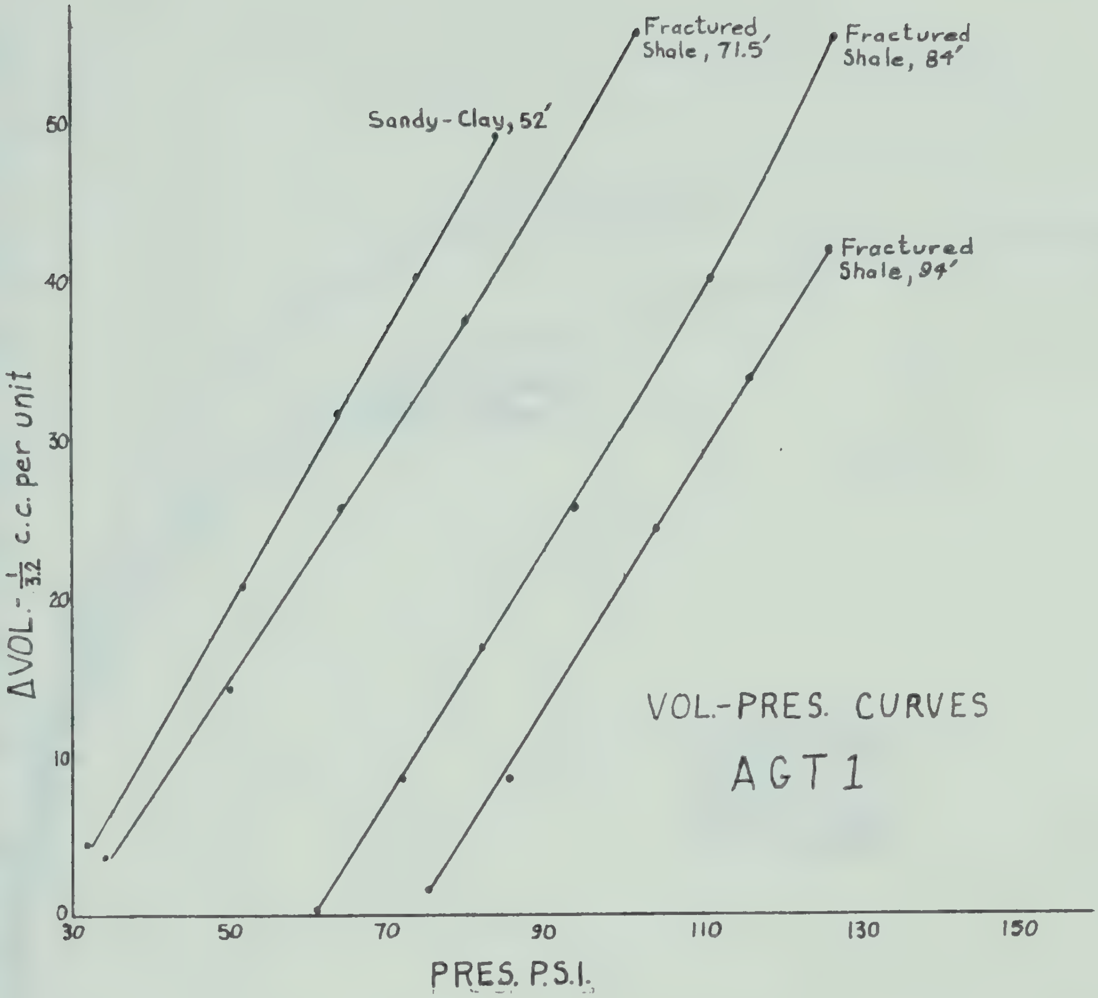
APPENDIX C

PRESSUREMETER

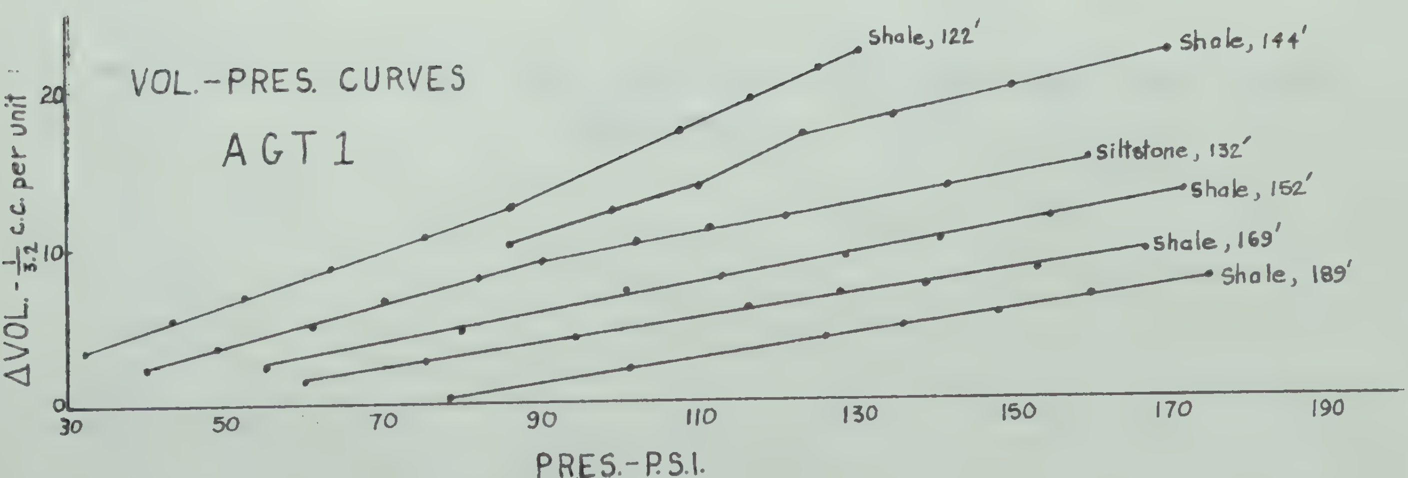
VOLUME-TIME AND

VOLUME-PRESSURE CURVES

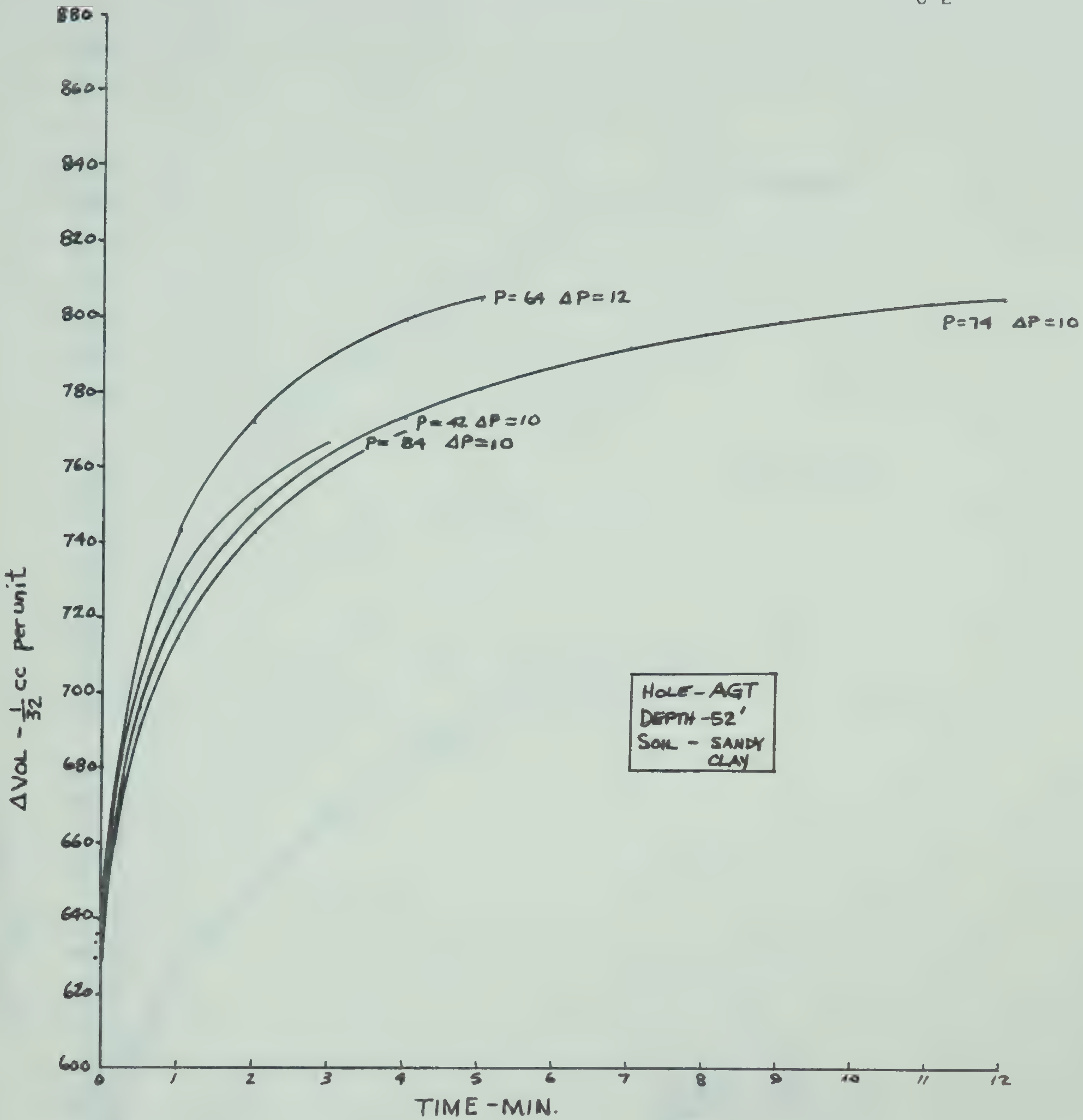
- AGT TOWER -

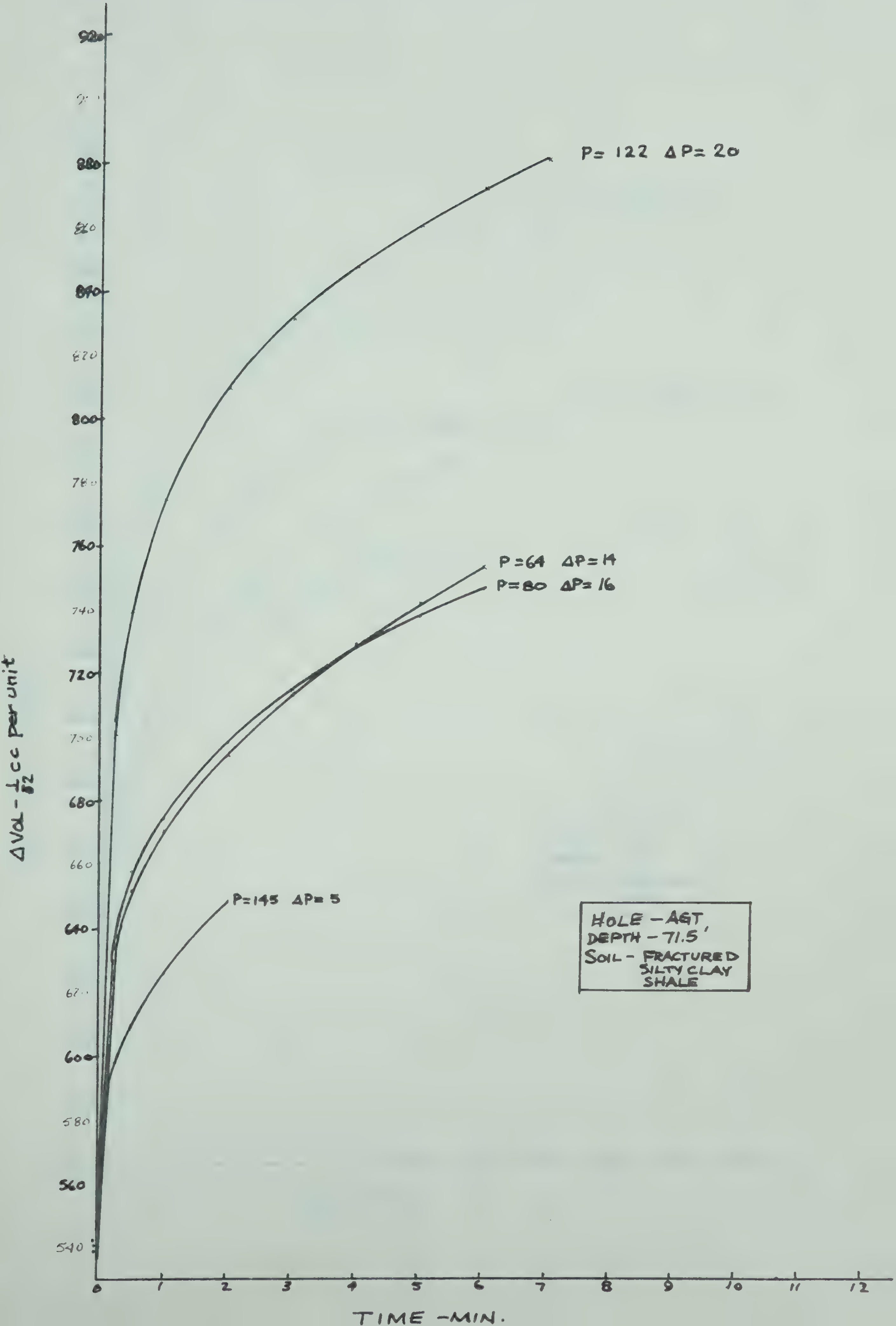


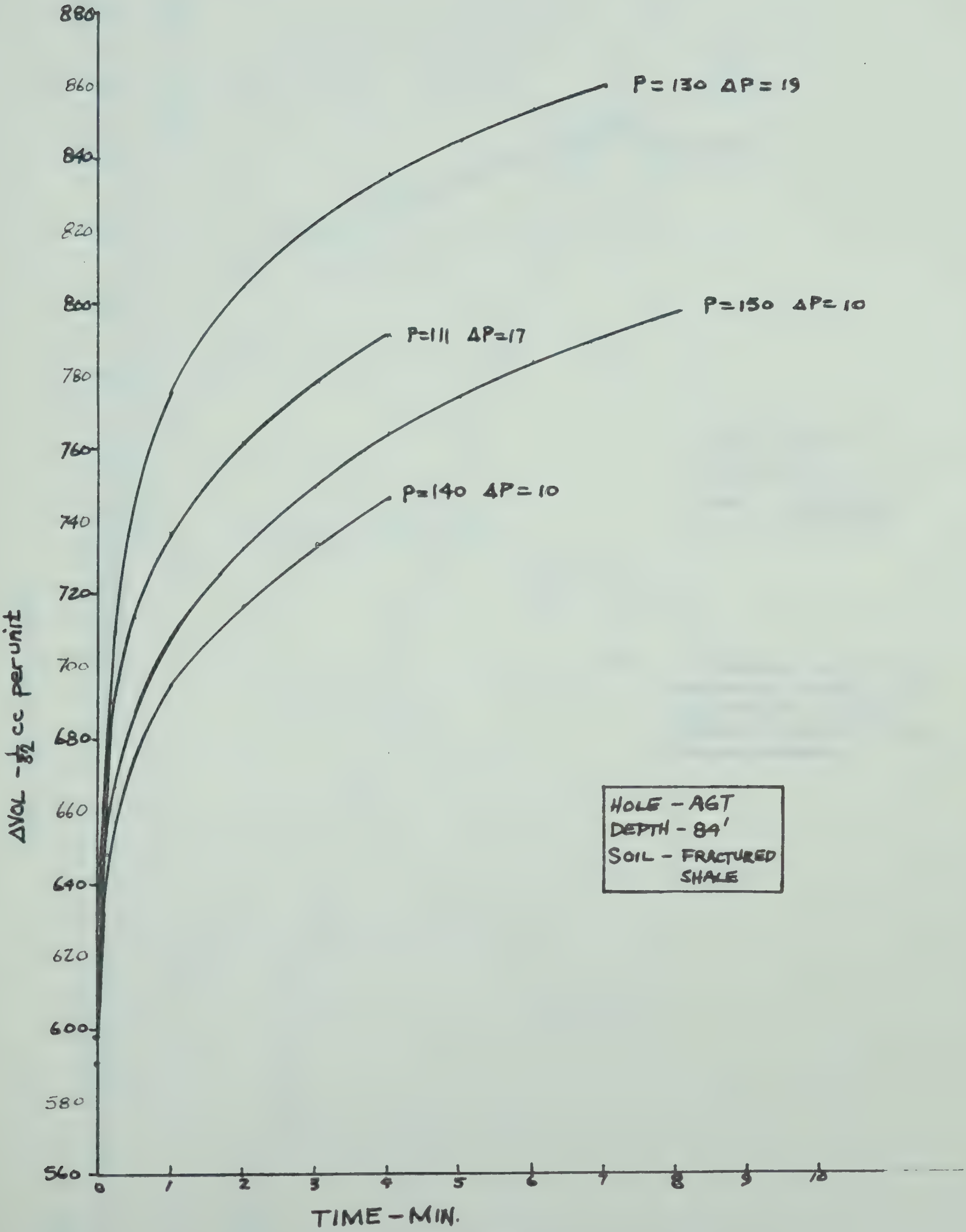
Fractured Shale, 71.5'
Fractured Shale, 84'
Sandy-Clay, 52'
Fractured Shale, 94'
VOL.-PRES. CURVES
AGT 1
ΔVOL. - 1/3.2 c.c. per unit
PRES. P.S.I.



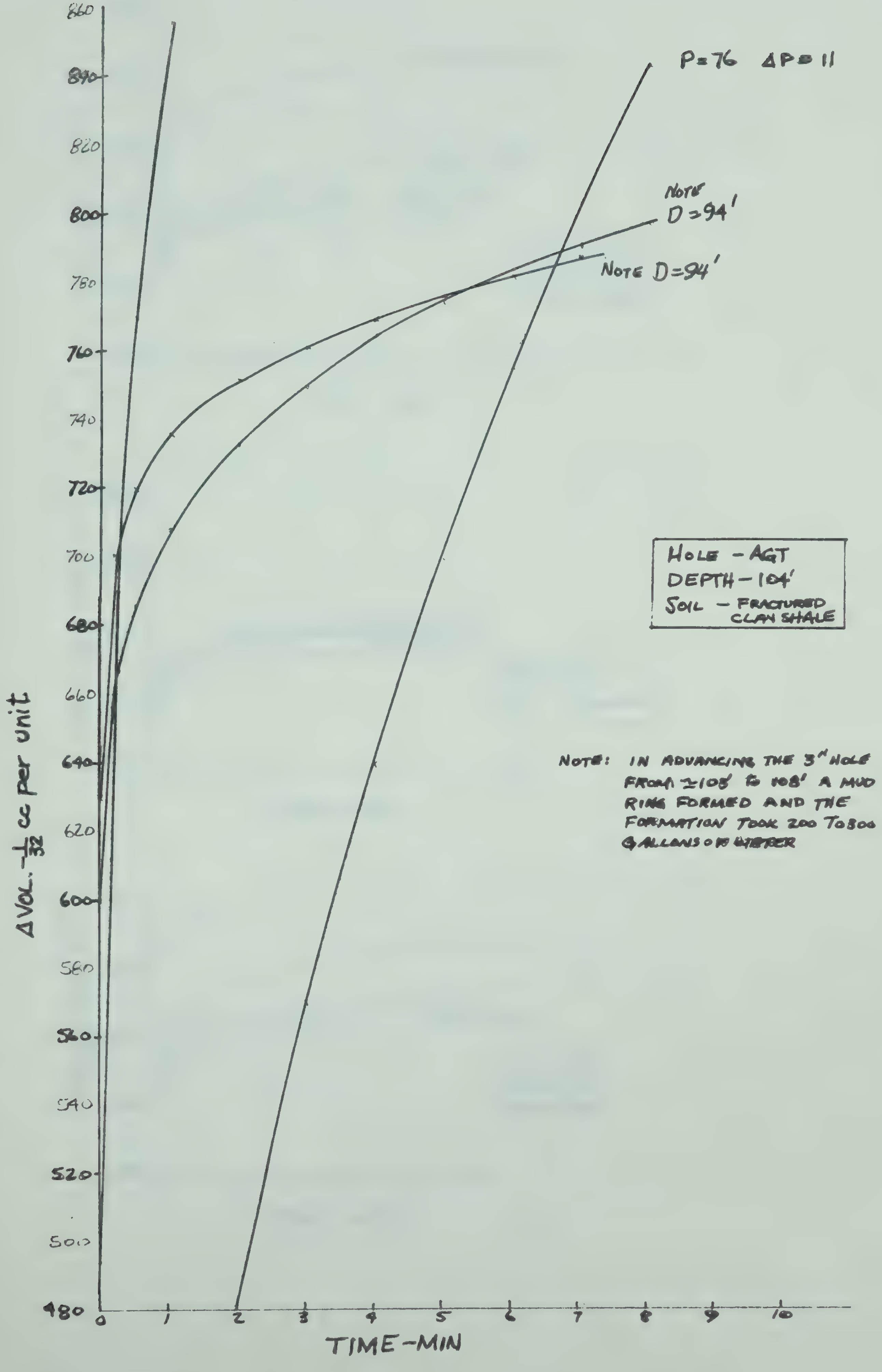
Shale, 122'
Shale, 144'
VOL.-PRES. CURVES
AGT 1
Siltstone, 132'
Shale, 152'
Shale, 169'
Shale, 189'
ΔVOL. - 1/3.2 c.c. per unit
PRES.-P.S.I.

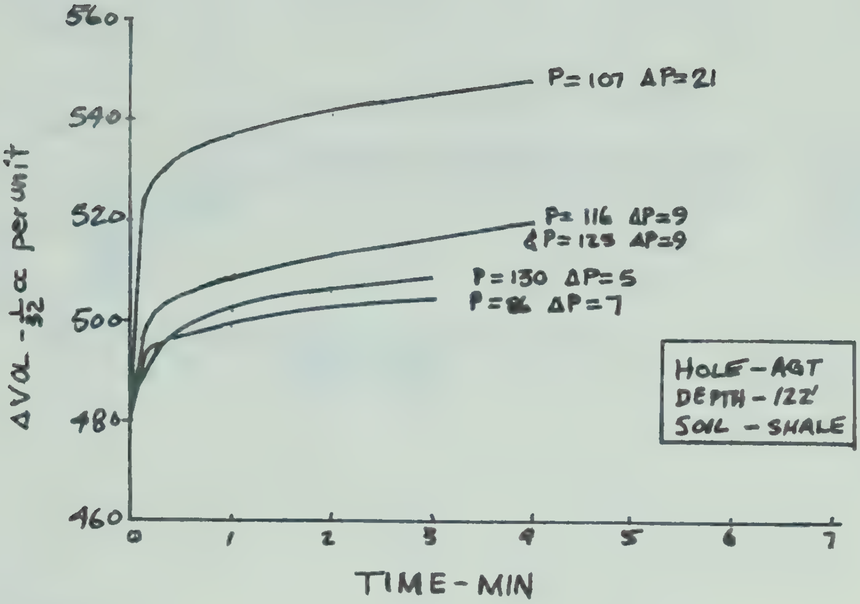

880
860
840
820
800
780
760
740
720
700
680
660
640
620
600
0
1
2
3
4
5
6
7
8
9
10
11
12
ΔVOL - 1/32 cc per unit
TIME - MIN.
P=64 ΔP=12
P=74 ΔP=10
P=42 ΔP=10
P=84 ΔP=10
HOLE - AGT
DEPTH - 52'
SOIL - SANDY CLAY


P= 122 ΔP= 20
P=64 ΔP=14
P=80 ΔP=16
P=145 ΔP=5
HOLE - AGT,
DEPTH - 71.5'
SOIL - FRACTURED SILTY CLAY SHALE
ΔVOL - 1/32 cc per unit
TIME - MIN.
880
800
760
740
720
680
660
640
600
580
560
540
0
1
2
3
4
5
6
7
8
9
10
11
12

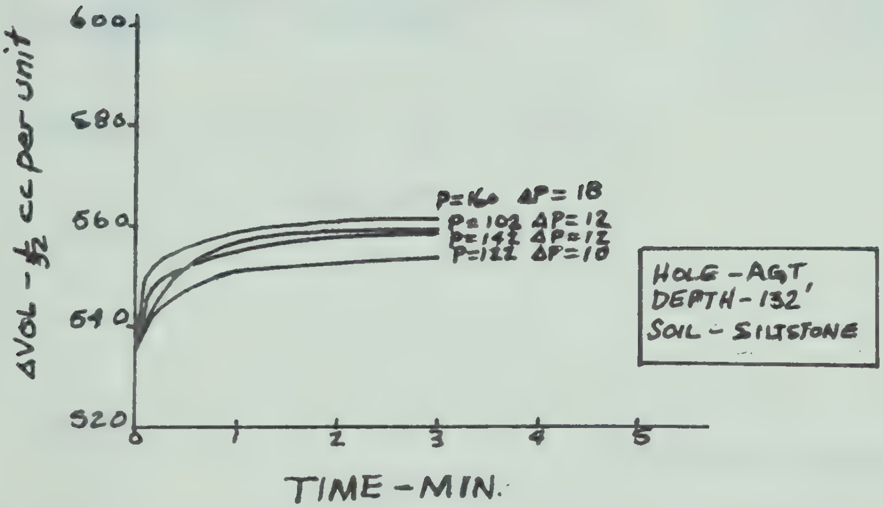


880
860
840
820
800
780
760
740
720
700
680
660
640
620
600
580
560
0
1
2
3
4
5
6
7
8
9
10
ΔVOL - 1/32 cc per unit
TIME - MIN.
P = 130 ΔP = 19
P=111 ΔP=17
P=150 ΔP=10
P=140 ΔP=10
HOLE - AGT
DEPTH - 84'
SOIL - FRACTURED SHALE

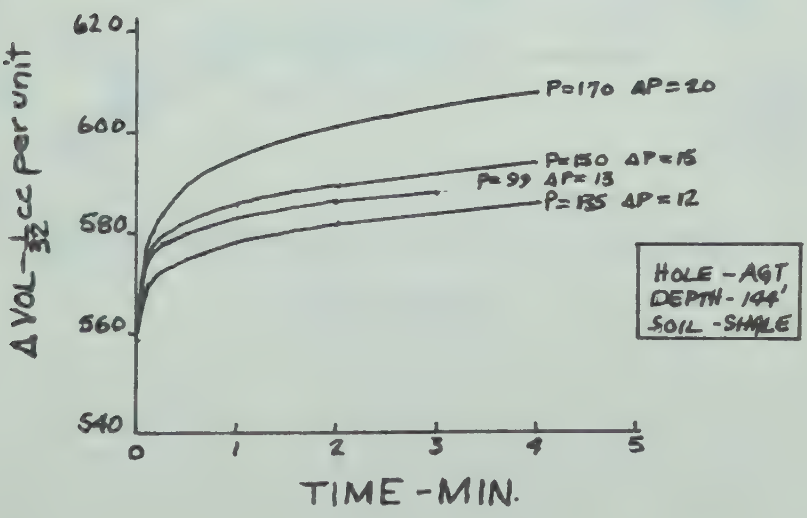

P=76 ΔP=11
NOTE D=94'
NOTE D=94'
HOLE - AGT
DEPTH - 104'
SOIL - FRACTURED CLAY SHALE
NOTE: IN ADVANCING THE 3" HOLE FROM ±105' TO 108' A MUD RING FORMED AND THE FORMATION TOOK 200 TO 300 GALLONS OF WATER
ΔVOL. - 1/32 cc per unit
TIME-MIN
480
500
520
540
560
580
600
620
640
660
680
700
720
740
760
780
800
820
840
860
0
1
2
3
4
5
6
7
8
9
10



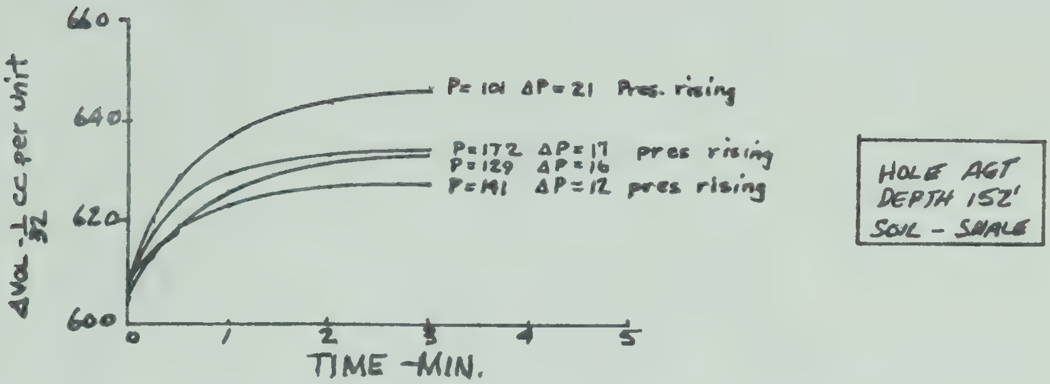
560
540
520
500
480
460
ΔVOL - 1/32 cc per unit
0 1 2 3 4 5 6 7
TIME-MIN
P=107 ΔP=21
P=116 ΔP=9
P=125 ΔP=9
P=130 ΔP=5
ΔP=7
HOLE-AGT
DEPTH-122'
SOIL - SHALE

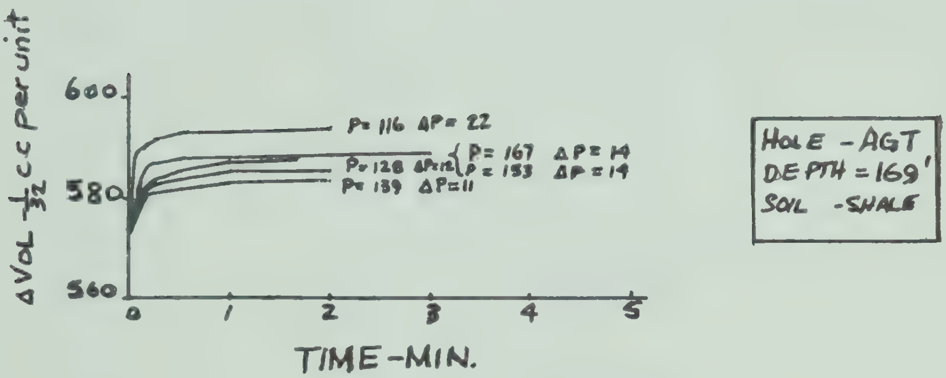


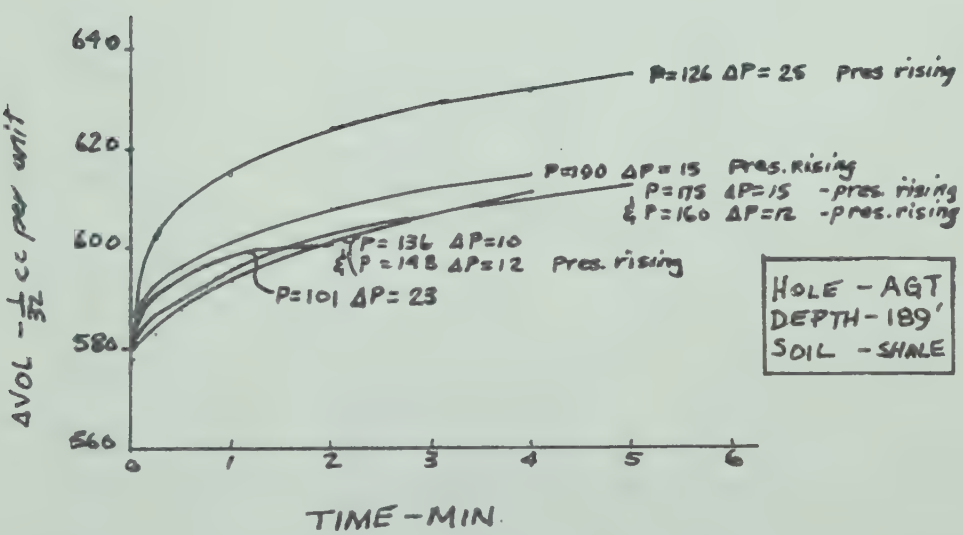
600
580
560
540
520
ΔVOL - 1/32 cc per unit
0 1 2 3 4 5
TIME-MIN.
P=160 ΔP=18
P=102 ΔP=12
P=142 ΔP=12
P=122 ΔP=10
HOLE-AGT,
DEPTH-132'
SOIL - SILTSTONE



620
600
580
560
540
ΔVOL-1/32 cc per unit
0 1 2 3 4 5
TIME-MIN.
P=170 ΔP=20
P=150 ΔP=15
P=99 ΔP=13
P=135 ΔP=12
HOLE-AGT
DEPTH-144'
SOIL-SHALE


660
640
620
600
ΔVOL - 1/32 cc per unit
P= 101 ΔP= 21 Pres. rising
P=172 ΔP=17 pres rising
P=129 ΔP=16
P=141 ΔP=12 pres rising
0
1
2
3
4
5
TIME -MIN.
HOLE AGT
DEPTH 152'
SOIL - SHALE


600
580
560
ΔVOL - 1/32 cc per unit
P= 116 ΔP= 22
P= 167 ΔP= 14
P= 153 ΔP= 14
P= 139 ΔP=11
0
1
2
3
4
5
TIME-MIN.
HOLE - AGT
DEPTH = 169'
SOIL -SHALE


640
620
600
580
560
ΔVOL - 1/32 cc per unit
P=126 ΔP= 25 Pres rising
P=190 ΔP= 15 Pres. rising
P=175 ΔP=15 -pres. rising
& P=160 ΔP=12 -pres. rising
P= 136 ΔP=10
& P = 148 ΔP=12 Pres. rising
P=101 ΔP= 23
0
1
2
3
4
5
6
TIME -MIN
HOLE - AGT
DEPTH - 189'
SOIL - SHALE

NOTATION

"A"-rod	1.3 inch diameter drill rod
Bx	Standard 2.25 inch diameter borehole size
CAU	Consolidated anisotropic undrained
CIU	Consolidated isotropic undrained
CU	Consolidated undrained
E	Modulus of deformation
ϵ_x	Strain in X-direction
γ_{ij}	Shear strain in the i-j-plane
Ko	Lateral stress ratio
μ	Poisson's ratio
Nx	Standard 2.94 inch diameter borehole size
Pi	Internal applied pressure
Po	Pressure level corresponding to full restoration of lateral in situ stresses
P_f	Pressure level corresponding to initiation of plastic behavior
Pu	Limit or ultimate pressure
r	Radius
σ_a	Axial stress
σ_r	Radial stress
σ_θ	Circumferential stress
SPT	Standard penetration test
Te	Elastic time interval - time interval during which measuring cell volume change at constant pressure reflects elastic deformation
3-D	Three-dimensional
τ_{ij}	Shear stress on i-plane in j-direction

2-D	Two-dimensional
UC	Unconfined compression
UU	Unconsolidated undrained
Vo	Volume of measuring cell at beginning of pseudo-elastic phase
V_i	Initial volume of measuring cell at beginning of pressuremeter test
V_i	Initial volume of measuring cell at the beginning of each pressure increment of the pressuremeter test